City of Waukesha, Wisconsin

130 Delafield Street │ Waukesha, WI 53188

Phase II Final Report
Sanitary Sewer System Master Plan

City of Waukesha
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City of Waukesha, Wisconsin

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EXECUTIVE SUMMARY

This report summarizes the results of the second of the two-phase Sanitary Sewer Master Plan. Building upon many of the findings of the first phase, this plan presents a variety of system improvements and capital improvement projects that will enable the City to provide reliable service for the next 20 years. In collaboration with Superior Engineering, Donohue has also prepared a CMOM Implementation Plan which provides specific operational and organizational improvements that will bring the City into conformance with EPA's CMOM guidelines (USEPA, 2005). This plan has been submitted under separate cover.

While the first phase focused primarily on evaluating system flows and capacities, this second phase focuses more on system condition and integrity. However it does expand on flow and system improvement analysis conducted under Phase I by including future flow projections and further refinement to improvement alternatives under consideration. The most important of these are interceptors intended to eliminate pump stations serving the western and southeastern portions of the sewer service area (SSA). Depending on the alternative selected, the pump station elimination force mains could cost anywhere from $18M - $41M.

The West-Side Interceptor could eliminate the following pump stations: Pebble Valley, Greenmeadow, Tallgrass, Summit, Heritage Hills, Coneview, Fiddler's Creek, and Badger Drive. The Southeast Interceptor could eliminate the following stations: Heyer Drive, Milky Way, West Avenue, and Burr Oak. These flows would be consolidated to the Fox Point pump station, which would have to be replaced. Replacing these stations with gravity sewers would improve system reliability by reducing the number of stations that would have to be maintained, would eliminate these stations' force mains, some of which have been problematic, and would reduce energy and O&M expenses.

Due to several failures and SSOs, EPA identified Waukesha's force mains as being in need of inspection. However since there is no effective way to visually inspect force mains, a desktop risk analysis was performed under Phase I of this project. This assessment ranked all force mains according to risk, the product of the likelihood and consequence of failure. The five highest risk force mains were selected for testing and inspection. These tests found little evidence of significant external corrosion. However Donohue recommends that Waukesha continue its program of replacing ferrous force mains with PVC force mains as streets are reconstructed.

A limited SSES was conducted in 2009 by smoke testing those areas that appeared to experience the most direct inflow. While only a small portion of the downtown area was tested in 2009, the number of defects per length of pipe was substantial. This may be due in large part to an average sewer age of 70 years. Therefore the smoke testing program was expanded in 2010 to include the entire downtown area. The results of both the 2009 and 2010 smoke testing are documented in this report. Over the course of the two-year program, 66 miles of sewer were tested (25% of all sewers owned by the City), and 41 defects were found, an average of only one defect every 1.5 miles of sewer.

With assistance from Visu-Sewer, Donohue performed visual inspections of 477 manholes in the downtown area for structural integrity. These inspections included the majority of manholes on

Waukesha's "30-Day List", those manholes particularly prone to accumulation of debris. 23% of the inspected manholes were found to be in poor to fair condition, with the remaining 77% in good to excellent condition. It will cost approximately $300K to rehabilitate all manholes up to excellent condition.

2008 pump run-time records indicated that the Heyer Drive pump station's sewershed experienced the worst base infiltration in the City. In 2009 a pump station monitor was installed to verify the pump run time findings, and concluded that base infiltration had actually increased from 2008 to 2009. Therefore in 2010 additional flow monitoring was conducted in the sewershed to better isolate the source of the infiltration. The majority of the I/I was found to be originating in the region north of the pump station, in the vicinity of where the Grey Terrace pump station had been. Donohue recommends that Waukesha televise the sewers in this area to locate and remedy the source of the infiltration.

Finally, in collaboration with the CMOM Plan and City personnel, three alternative versions of a 20-year Capital Improvement Plan (CIP) ranging from $65M - $95M have been prepared. While subject to change and refinement, the selected CIP plan should enable Waukesha to plan and budget for system improvements required through the year 2035.

CHAPTER I – INTRODUCTION

This report documents the completion of the second of this two-phase master planning effort. A draft of the first phase of this master plan was submitted to Waukesha, DNR, and EPA in June, 2010.

1.1 PROJECT BACKGROUND

On May 13, 2008 and August 26, 2008, the U.S. Environmental Protection Agency conducted inspections of Waukesha's sanitary sewer system. In October 2008, EPA sent two letters to Waukesha that specified that in addition to providing additional information to EPA, Waukesha was to perform the following tasks:

- Conduct a Sanitary Sewer Evaluation Study (SSES) for the entire sanitary system;
- Conduct an assessment of all force mains;
- Investigate sources of inflow and infiltration (I/I) and develop mitigation plan;
- Eliminate constructed relief points at the Coneview and Burr Oak pump stations;
- Develop a Capacity, Management, Operations, and Maintenance (CMOM) program.

In January 2009, Waukesha began a 2-year effort to prepare a Sanitary Sewer Master Plan which would address the preceding requirements. While a CMOM Implementation Plan has been developed as part of this study, the CMOM plan is intended to be a stand-alone document and is being submitted under separate cover.

1.2 PROJECT OBJECTIVES

The objectives of the first phase of this project were:

- Conduct a Force Main Risk Assessment Desktop Evaluation;
- Develop alternatives to protect pump stations from flooding;
- Develop a collection system model;
- Identify hydraulic deficiencies;
- Conduct flow monitoring;
- Quantify inflow and infiltration;
- Develop pump station elimination alternatives;
- Perform limited smoke testing;
- Evaluate CMOM requirements and develop program plan.

This second phase builds upon many elements of the first phase. The objectives of this second phase of the master planning project are:

- Perform indirect and visual inspections of highest risk force mains;
- Conduct follow-up flow monitoring of the Heyer Drive pump station sewershed;
- Conduct smoke testing of the downtown area;
- Inspect 500 manholes;
- Estimate future flows;
- Refine pump station elimination alternatives;
- Develop a 20-year Capital Improvement Plan.

CHAPTER II –FORCE MAIN EXTERNAL CORROSION DIRECT ASSESSMENT

2.1 BACKGROUND

In 2008, EPA mandated that Waukesha "Complete an assessment of the condition of each force main in Waukesha's collection system, taking into account… [material, age (or installation date), diameter, length, capacity, typical flow rates, cathodic protection (type and current condition), and inspection and maintenance history] as well as structural integrity and stability." This assessment is intended to reduce the risk of future force main failures and SSOs.

In 2009, Waukesha completed a Force Main Risk Assessment, ranking each force main according to risk (Table 1), which is the product of the likelihood and consequences of failure. This assessment was included in the Draft Phase I Sanitary Sewer Master Plan submitted in June 2010[1]. The five highest risk force mains were selected for visual inspection; they are listed below and shown in Figure 1.

- West Ave[2] – From S West Ave. & Dodie to S West Ave. & Baird
- Burr Oak – From Burr Oak Blvd east of Oakdale to Burr Oak Blvd & Chapman
- Heyer Dr – From Heyer Dr & Larchmont Dr to E Sunset Dr & Tenny Ave
- Greenmeadow – From park, 640' west of Grandview Blvd to Joellen Drive
- Pebble Valley – From Broken Hill Ct & Hunting Ridge Rd to Emslie Dr & Northview Rd

As indicated in the force main risk assessment, external corrosion is the primary cause of force main failure. Therefore, External Corrosion Direct Assessment (ECDA) was selected as the best available inspection methodology.

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[1] Donohue & Associates, *DRAFT – Final Report, Phase I Sanitary Sewer Master Plan*, 2010.
[2] In 2010, approximately 1,500 feet of force main was replaced. It is likely that the West Ave Pump Station will be eliminated as part of the proposed southeast sanitary interceptor sewer project; therefore, this force main was removed from the list of those to be tested.

Table 1 – Force Main Risk Assessment

Force Main	Likelihood	Consequence	Risk	Material	Age	Length
West Avenue	3.02	3.12	9.40	CI	53.5	3301
Greenmeadow 2 (ends 594' from Greenmeadow 1)	2.23	3.50	7.82	DI	42.0	594
Pebble Valley	1.79	4.20	7.54	DI	43.5	4154
Heyer Dr 2 (ends 1822' from Heyer Dr 1)	1.96	3.70	7.26	DI	43.5	1822
Burr Oak Boulevard 1 (ends 2004' from PS)	2.14	3.16	6.76	DI	41.2	2004
General Electric	2.48	2.53	6.29	DI	27.7	5034
Greenmeadow 1 (ends 924' from PS)	1.80	3.50	6.28	DI	12.4	924
Greenmeadow 3 (ends 1945' from Greenmeadow 2)	1.24	4.40	5.45	DI	27.9	1945
Coneview	1.51	3.55	5.36	DI	34.5	2563
Heyer Dr 1 (ends 835' from PS)	1.49	3.55	5.30	DI	17.4	834
Greenmeadow 4 (ends 2327' from Greenmeadow 3)	1.24	4.20	5.20	DI	25.7	2327
Ruben Drive 1 (ends 1524' from PS)	1.64	3.11	5.08	DI	24.2	1524
Burr Oak Boulevard 2 (ends 3538' from Burr Oak 1)	1.55	3.19	4.94	CI	43.9	3538
Northview Road	1.72	2.77	4.77	CI	43.5	713
Milky Way 3 (ends 124' from Milky Way 2)	1.55	3.06	4.76	CI/DI	37.9	124
Wal-Mart	1.63	2.87	4.67	DI	21.4	1201
Sunset Drive	1.76	2.67	4.70	CI	47.6	3831
Badger Dr 1 (ends 1305' from PS)	1.90	2.31	4.38	DI	29.4	1305
Milky Way 1 (ends 814' from PS)	1.41	3.06	4.33	PVC	21.1	814
Milky Way 6 (ends 242' from Milky Way 5)	1.41	3.06	4.33	PVC	21.1	242
Milky Way 4 (ends 41' from Milky Way 3)	1.39	3.06	4.24	CI/DI	26.4	41
Greenmeadow 5 (ends 3940' from Greenmeadow 4)	1.09	3.83	4.18	DI	17.0	3940
Milky Way 2 (ends 31' from Milky Way 1)	1.34	3.06	4.09	CI/DI	23.1	31
Milky Way 5 (ends 25' from Milky Way 4)	1.34	3.06	4.09	CI/DI	23.1	25
Fox Point	1.15	3.55	4.09	PVC	25.4	8160
Ruben Drive 3 (ends 3850' from Ruben Drive 2)	1.23	3.24	4.00	DI	27.7	3850
MacArthur Road	1.30	2.99	3.89	DI	22.7	2279
Ruben Drive 2 (ends 1137' from Ruben Drive 1)	1.22	3.11	3.80	DI	27.0	1137
Springbrook	1.26	2.94	3.71	DI	18.4	4056
Corporate Drive 2 (ends 1323' from Corporate Dr 1)	1.38	2.47	3.41	PVC	14.4	1323
Corporate Drive 1 (ends 3937' from PS)	1.81	1.78	3.21	PVC	10.4	3937
Summit Avenue	1.11	2.71	3.00	DI	14.4	2324
Hollidale	1.20	2.50	3.01	CI	29.4	68
Woodfield	1.10	2.49	2.73	DI	25.0	701
Corporate Drive 3 (ends 411' from Corporate Dr 2)	1.17	2.29	2.67	PVC	10.4	411
Wesley Drive	1.05	2.47	2.60	PVC	11.4	1682
Dana (River Hills)	1.02	2.48	2.53	PVC	10.4	1546
Aviation Drive	1.08	2.25	2.43	PVC	13.44	4980
West Bluemound	1.15	2.03	2.32	PVC	12.4	4732
Badger Dr 2 (ends 3385' from Badger Dr. 1)	0.98	2.31	2.27	HDPE	3.0	3385
Heritage Hills (Madison Street)	0.76	2.92	2.23	PVC	7.4	1816
Tallgrass	0.97	2.07	2.01	PVC	14.1	1335
Silvernail	0.89	2.18	1.94	PVC	10.4	3054
Fox Lake Village	0.75	2.48	1.85	HDPE	5.4	3960
Deer Path	0.74	2.47	1.82	PVC	10.4	1093
Bluemound	1.19	1.48	1.76	DI	31.5	516
River Place	0.58	3.01	1.75	PVC	18.4	405
Rivers Crossing 1 (ends 1217' from PS)	0.53	2.52	1.33	PVC	12.4	1217
Rivers Crossing 2 (ends 2649' from River Crossing 1)	0.40	2.52	1.00	PVC	3.6	2649
Fiddlers Creek	0.40	2.45	0.99	PVC	10.4	1025
Golf Road	0.39	2.46	0.95	PVC	28.4	1474
Pearl Street 1 (ends 788' from PS)	0.35	2.31	0.81	PVC	3.4	788
Pearl Street 2 (ends 648' from Pearl Street 1)	0.33	2.31	0.77	PVC	2.4	648
Deer Trails	0.03	2.45	0.07	PVC	4.4	800

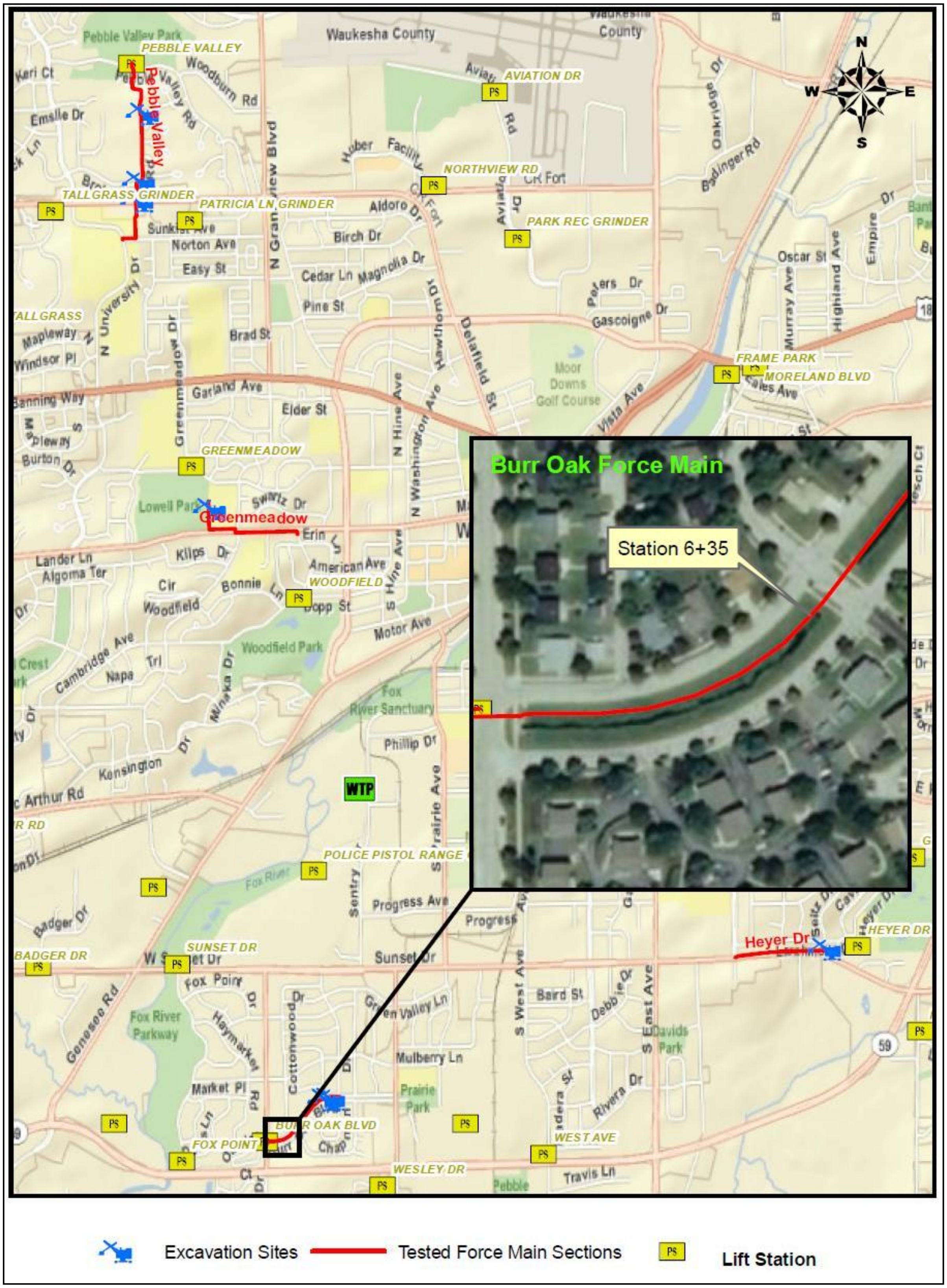


Figure 1 - Testing Locations

2.2 ECDA METHODOLOGY

The National Association of Corrosion Engineers (NACE), under a directive from the U.S. Government, recently developed a methodology for assessing and reducing the impact of external corrosion on the integrity of onshore buried pipelines (primarily ferrous pipelines). The methodology is termed an External Corrosion Direct Assessment (ECDA). ECDA is an evaluation technique developed since 2002, from both long utilized corrosion monitoring techniques and recently developed measurement technologies, primarily for use in the gas and chemical pipeline industries. The methodology is approved by DOT 49 CFR Part 192 to assess external corrosion. ECDA is a continuous improvement process designed to not only identify areas where external corrosion is underway, but to also predict potential future corrosion areas, which will assist greatly in future corrosion prevention. Since the majority of Waukesha force main failures were the result of external corrosion, ECDA is the best technology available for evaluating those force mains at greatest risk.

ECDA is a two-step process. First, a series of indirect electrical potential measurements are made at ground surface to locate areas of possible corrosion, called “indications.” Secondly, some of these indications are excavated and visually examined to determine the actual extent and severity of corrosion. The directly observed pipeline condition is then extrapolated to the remainder of the pipeline based upon the indirect measurements. Although this extrapolation is inferential, it has proven very useful for management of corrosion of underground pipelines.

2.2.1 INDIRECT INSPECTION

Indirect measurement was performed using three techniques. Soil resistivity was measured at various points along the force mains. The presumption is that all other conditions being equal, corrosion will be more severe in those areas of lower soil resistivity than in those areas of higher soil resistivity. Areas of probable corrosion were located by one of two electrical potential measurement techniques.

Both techniques only indicate the <u>likelihood</u> of corrosion rather than the actuality. Furthermore, the magnitude of the measurement, while related to the severity of corrosion, may be influenced by many extraneous factors. The locational precision of the measurements are impacted by both the electrode separation and the depth of cover. In addition, when utilizing the two-cell technique, there is no direct connection to the pipeline, therefore the measured electrical field may be caused by corrosion of a nearby foreign structure. Once the indirect measurements were reviewed, those sites with the greatest corrosion potential were selected for excavation and direct examination. Sites were selected based on the intensity and clarity of the indication relative to the remainder of the pipeline.

The data from the direct examinations was reviewed and then extrapolated to the remainder of the force mains. This extrapolation was done by comparing the area under the anodic portion of the potential survey curves. The methodology is based on the fact that the intensity of the corrosion will affect both the intensity of the measured potential and the distance over which it is apparent. It should be noted that this extrapolation is not definitive, since various underground conditions can alter the measured values, but it does provide a reasonable first approximation of the overall pipeline condition. Please note that the values presented are estimated based on a limited data set and should not be relied upon as if they were actual measurements. Despite these limitations, aside from exposing the entire pipeline, this is the most reliable method of assessing pipeline condition.

2.2.1.1 Close-Interval Potential Survey (CIS)

Where the force main was "electrically continuous" a Close-Interval Potential Survey (CIS) was performed. With this technique, a connection is made to the force main and the electrical potential between the pipeline and a copper/copper sulfate ($Cu/CuSO_4$) reference electrode placed on the ground surface at short intervals over the pipeline centerline is measured. The more electro-negative (anodic) the measured potential, the greater the probability that corrosion is occurring. Where the force main is not electrically continuous, a CIS cannot be performed and a cell-to-cell survey was performed.

In the Cathodic Protection industry, it is well known that pipe-to-soil potential (voltage) measurements at test stations, which are typically spaced a considerable distance apart, are insufficient to judge the overall condition of a pipeline and to judge whether or not there is complete protection. As a result, close interval potential surveys involving the measurement of potentials at closely spaced intervals along the entire length of a pipeline have become the industry standard. In fact, with regard to the ECDA protocol, pipe-to-soil potential readings are typically recorded at 2.5-foot intervals between test stations. (Test stations are insulated electrodes that are in permanent electrical contact with the pipeline and that can be contacted above ground.)

Figure 2 illustrates the essential components of a close interval potential survey measurement apparatus. The key components of the CIS apparatus are the reference electrode, connecting the negative terminal of the data logger to the soil, and the data logger, the positive terminal of which is connected to the test station (and therefore to the pipe). The Data Logger is a sophisticated digital voltmeter/data storage unit. With this apparatus, the potential difference (voltage difference) between the pipe and the soil (at the reference electrode location) can be measured and this data point (voltage at a specific location along the length of the pipe with respect to the reference electrode) can be stored for processing by means of the digital voltmeter/data storage unit (data logger). Similarly, data (of pipe-to-soil potential with respect to the reference electrode) can be recorded at intervals of, typically, 2.5 to 5.0 feet, along the length of the pipeline.

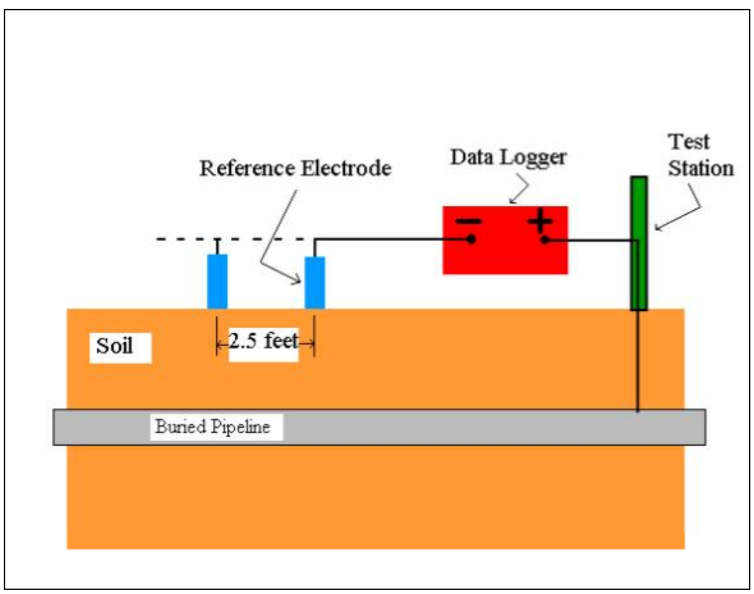


Figure 2 – Key Components of a CIS Test Equipment System

The ultimate goal of a CIS is to identify locations (if any) along the length of a buried pipeline that are not registering a sufficient potential difference between pipe and soil, which would be indicative of locations that might be experiencing external corrosion. There is an industry standard (0.85 Volts or 850mV) which is applied in the Cathodic Protection industry and which represents the minimum potential difference (voltage) recorded between the pipe and the soil (with respect to a particular reference electrode that signifies sufficient cathodic protection. Since, on a pipeline that is under cathodic protection (CP) (impressed current CP), the pipe is held at negative potential due to an electron current flowing to (and in) the pipe, the minimum potential difference between pipe and soil would be -0.85 Volts. Any more-positive (less-negative) voltages, for example, -0.7 Volts, would suggest insufficient cathodic protection and would indicate a location where external corrosion might be taking place. Actually, a measured potential difference of greater than -0.85 Volts (for example, -0.95 Volts or higher) will be required to be in the "safe" area with regard to a pipeline being fully cathodically protected under current flow conditions, particularly if the amount of ionic current flowing in the soil to the pipe is large. This situation would result in a significant, so-called, IR drop (voltage drop) due to the ionic current flow which must be added to the minimum 0.85 Volt potential difference to ensure sufficient cathodic protection. It is possible to determine the magnitude of this IR drop voltage during the performance of a CIS by conducting the CIS measurements in the "High-Low", or current-interrupted, mode, where the pipe-to-soil potential is sampled as the current is switched ON and OFF in a cyclic fashion. The critical pipe-to-soil potential (with regard to ensuring sufficient cathodic protection) would be the potential measured during the current OFF part of the cycle, since in this case the IR drop would be eliminated.

In any case, once a critical pipe-to-soil potential difference has been established for sufficient cathodic protection of a particular pipeline (taking the IR drop into account), a CIS can be performed to monitor the condition of the pipe, by comparing the pipeline potential profile recorded with the ideal case scenario, which would be a uniform (constant) potential along the length of the pipe.

2.2.1.2 Cell-To-Cell ("Surface Potential") Survey

For those force mains that were found to be electrically discontinuous, a two-cell potential survey was performed. This technique measures the electrical field produced by corrosion between two $Cu/CuSO_4$ reference cells placed a fixed distance apart (typically 20') over the force main centerline. Probable corrosion is indicated by a reversal in the polarity of the electrical field.

For these surveys, any localized current flow that gives rise to potential gradients on the surface of the soil above a buried pipe is due to the presence of corrosion cells (combinations of anodic and cathodic areas) on the pipeline, as opposed to impressed current from a CP system which is responsible for the "signal strength" in the case of CIS surveys.

In the case of bare pipe, typically only about 10-15 % of the pipe will be subject to galvanic corrosion and, in addition, typically this small percentage is made up of small, highly localized, corrosion areas (anodic areas) that are randomly distributed along the length of the pipe. Thus, an "above-the-ground" survey technique that can accurately locate these isolated areas is invaluable.

The objective of SP surveys is to locate anodic areas existing along a segment of pipeline, as evidenced by potential gradient fields presenting themselves on the surface of the soil directly above the anodic areas. Once any anodic areas have been located, remedial action can be taken, such as the installation of “sacrificial” anodes to suppress current flow from the corroding area, with a view to preventing further external corrosion in that particular area.

Referring to Figure 3, when current flows onto (or away from) a localized area on a buried-pipeline, a voltage gradient field presents itself on the surface of the soil directly above the localized area. In the case where current is flowing onto a pipeline at some localized area, that localized area is considered a cathodic area and the voltage gradient field on the soil above the pipe will have a negative polarity. The largest negative potential will exist directly above the anomaly and the negative potential will decrease in magnitude to remote earth potential with distance away from the pipe.

The opposite is true in the case where current is flowing away from an isolated (localized) area on a buried-pipeline. In this case, the area is considered an anodic area and the voltage gradient field presenting itself on the surface of the soil above the pipe will have a positive polarity. The largest positive potential will exist directly above the anodic area and the positive potential will decrease in magnitude to remote earth potential with distance away from the pipe.

Since corrosion occurs on an uncoated buried pipeline via the development of “corrosion cells”, both anodic and cathodic areas must exist simultaneously. The current flowing away from the anodic area will be collected by the cathodic area and the return path for the current will be the pipeline itself as illustrated below.

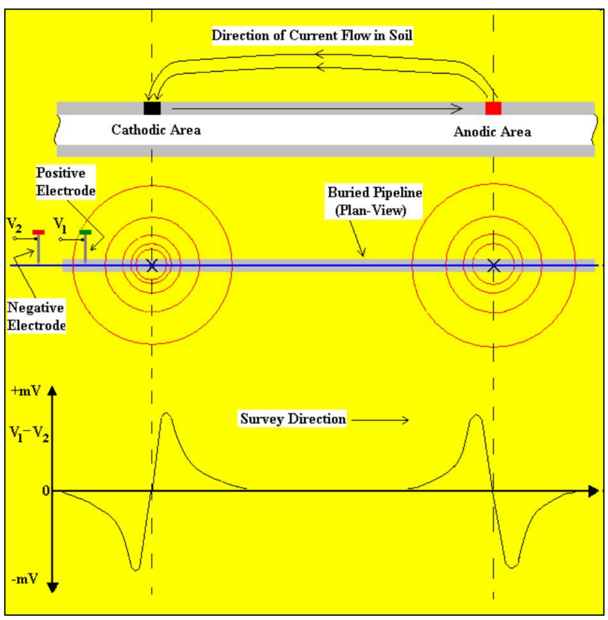


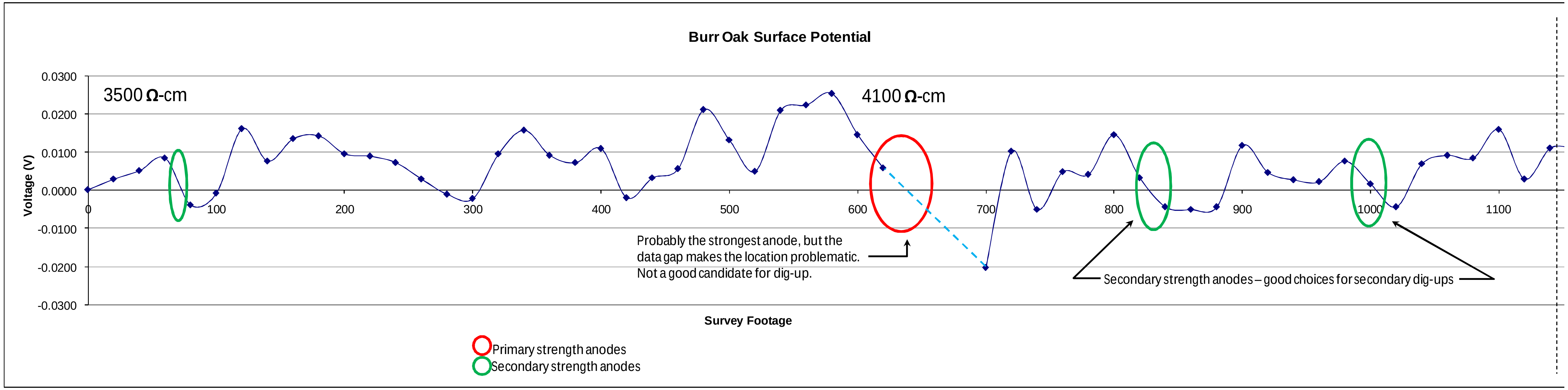
Figure 3 – In-Line method of conducting SP surveys

2.3 BURR OAK

The Burr Oak force main was tested from Station 0+00 to Station 20+40, utilizing the Two-Cell Potential Profile technique. Soil resistivity was measured at three locations (Stations 0+00, 6+80 & 15+80). These results are illustrated in Figure 4. Soil resistance ranged from 3500 Ω-cm to 4600 Ω-cm; these values are generally considered moderately corrosive. Two-cell potential measurements ranged from +0.0254V to -0.0202V. There are thirteen identifiable anode/cathode pairs in the measurements. Of these anodic locations, nine were isolated as potentially useful for direct examination. Three of these sites were noted as primary sites (circled in red), those with the largest potential differential, clearest indication of an anodic location, and highest probability of significant corrosion. The other six sites were identified as secondary sites (circled in green), sites of lesser intensity, less clearly anodic locations, and probably experiencing lesser corrosion, but potentially useful for comparison.

Based upon field conditions, two sites were selected for excavation and direct examination. These sites, located at stations 16+80 and 18+20 are indicated on Figure 4 by vertical red arrows. The soil found at pipe depth appeared to be approximately equal parts sand, gravel and clay, typical of the migration of native clay soil into select granular backfill over time. The soil near the pipeline was moist, and the measured soil resistivity far lower (600 Ω-cm & 800 Ω-cm) than that measured at grade. This would suggest that corrosion of the pipeline would likely be more severe than would have been suspected from the resistivity measured at grade. The force main surface was found to be covered with a black scale, which may have some protective value. Beneath the scale some scattered pitting was found, with diameters up to ½" and depths to 50 mils. This level of corrosion on a pipeline approximately forty-three years old would be considered minor and does not compromise pipeline integrity.

Each of the thirteen anodic indications was evaluated to determine the apparent relative size of the anode. The potential difference between the measured electro-positive peak prior to the potential reversal and the electro-negative valley following the reversal, and the distance between these points was used to calculate the triangular area beneath this portion of the Two-cell Potential Survey curve. Each of these apparent anode sizes was then compared to the apparent anode sizes at the two directly examined sites, and a relative estimated maximum depth of pitting was calculated. Given an installation date of 1967 and original pipe wall thickness of 0.380", these were converted to an estimated average annual pitting rate and estimated minimum remaining wall thickness. These are illustrated in Figure 5. The measured maximum pit depth of 0.05" at both direct examination sites translates to an average pitting rate of 1.2 mils/yr. Based upon the apparent relative anode sizes, the worst corrosion is occurring at Station 6+35 (Figure 5) adjacent to Cottonwood Drive (Figure 6). Locations adjacent to road crossings commonly have higher corrosion rates. The estimated maximum depth of penetration at this location would be 160 mils, translating to an average pitting rate of 3.7 mils/yr and a minimum remaining wall thickness of just less than 58 percent. This appears to be an area that should be re-tested every 5-10 years. Should conditions remain the same, and barring structural failure due to pipe wall thinning, the estimated pitting rate would produce a pipe wall penetration in approximately sixty years.



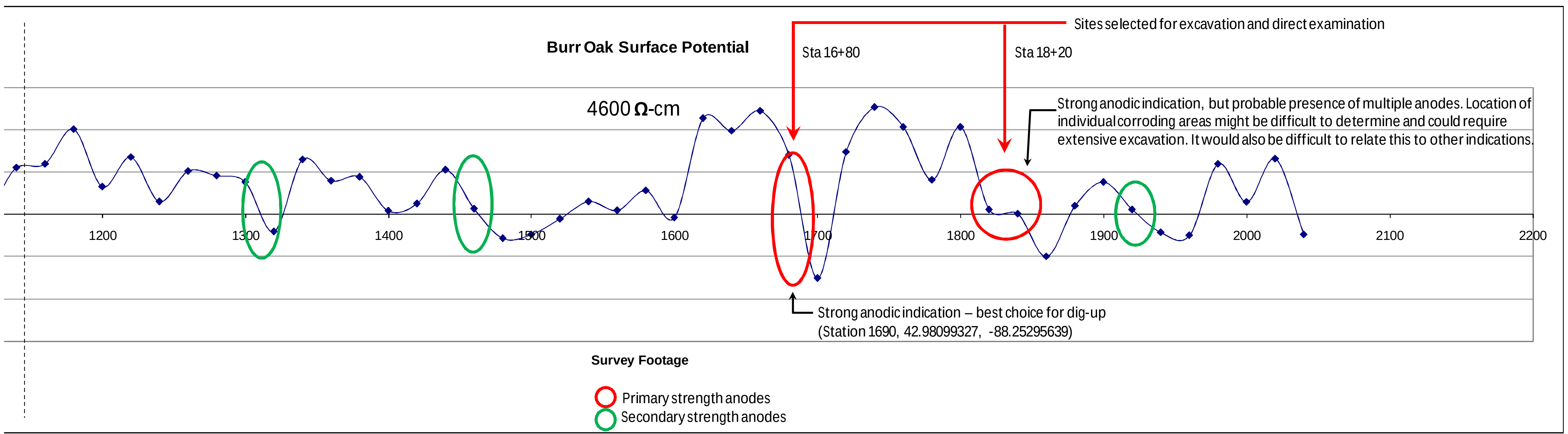


Figure 4 – Burr Oak Surface Potential

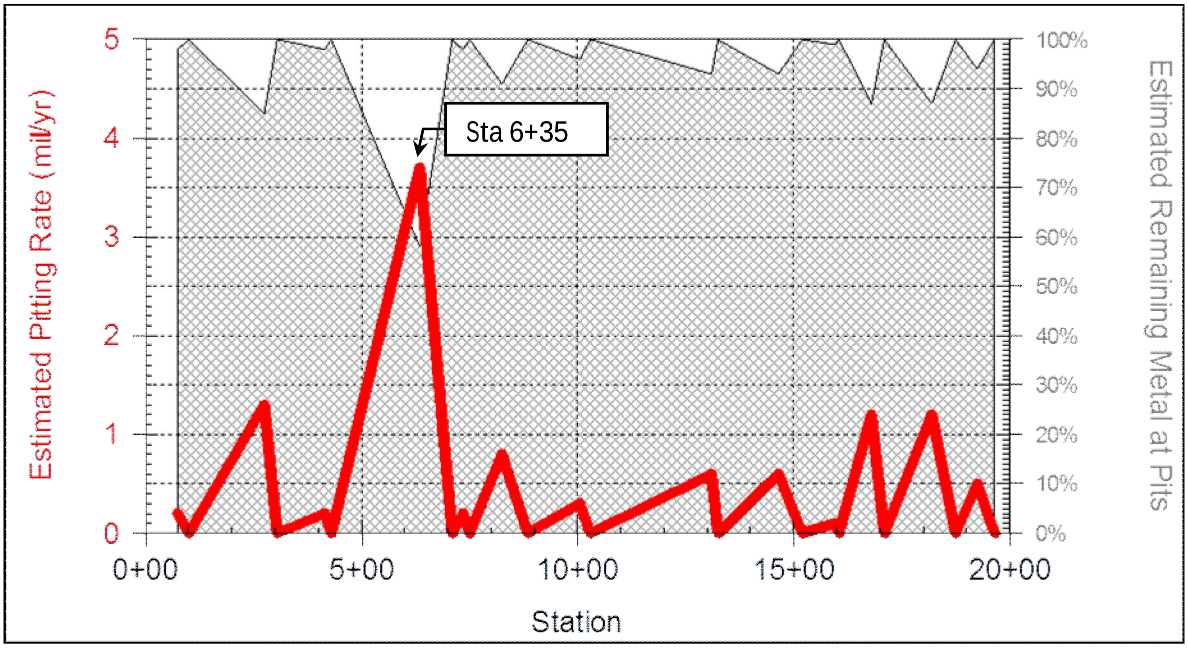


Figure 5 – Burr Oak Force Main Estimated Condition

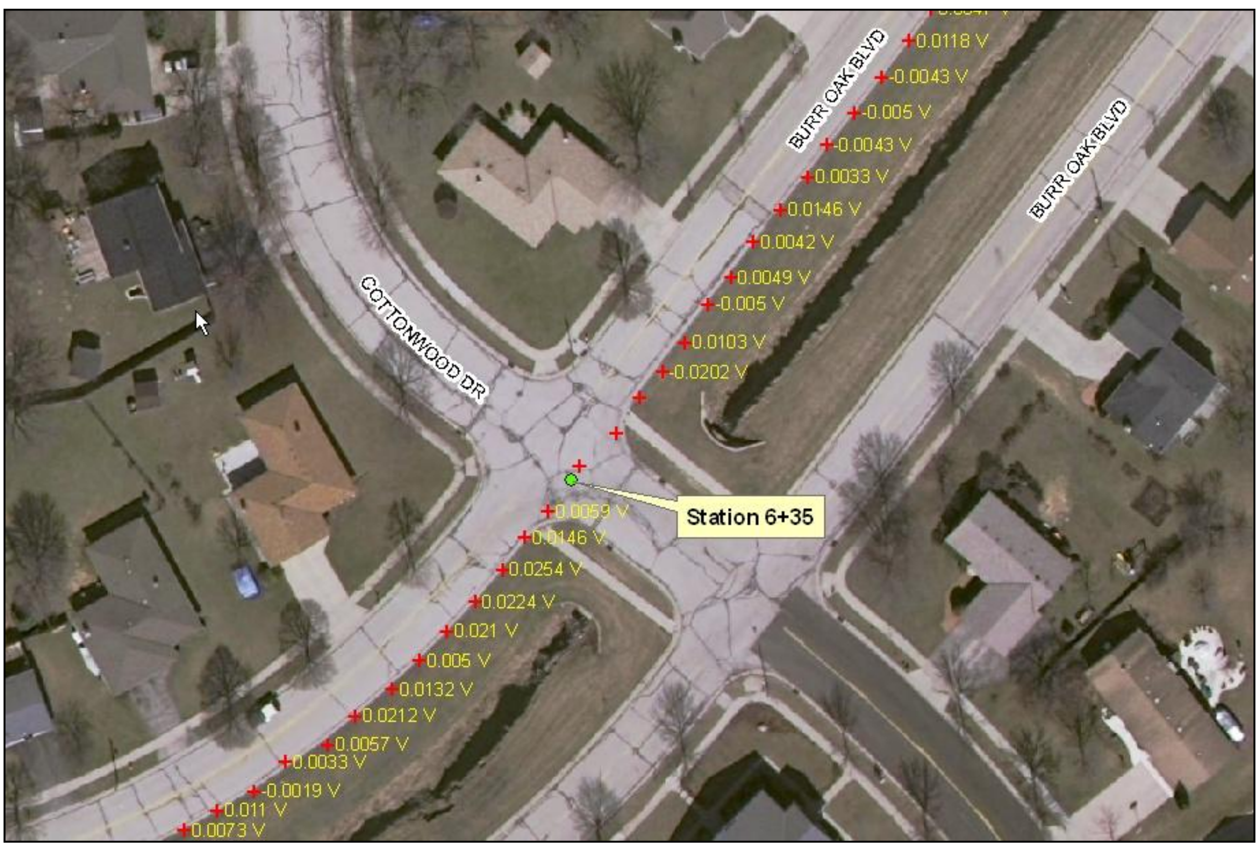


Figure 6 – Location of Possible Burr Oak Pipe Thinning

At each of the direct examination sites, a galvanic anode was connected to the exposed pipeline. This should protect the individual pipe stick at this location and preserve it from further corrosion for at least the next twenty years. Unfortunately, since this force main is electrically discontinuous, the anode will have minimal, if any, effect on the remainder of the pipeline.

2.4 GREENMEADOW

The Greenmeadow force main was tested between stations 0+00 and 28+55. This force main was electrically continuous; therefore, a CIS was performed (results in Figure 7). Soil resistance was measured at two locations, Station 6+15 and Station 9+03 with readings of 16,900 Ω-cm and 1900 Ω-cm, respectively. At the higher soil resistivity, little or no corrosion would be expected. This is consistent with its proximity to the most cathodic measured pipe-to-soil (P/S) potential of -0.2628V at Station 5+60. Conversely, a soil resistivity of 1900 Ω-cm would be considered corrosive, particularly if there is a rapid transition from a much higher soil resistivity (as in this case). This is consistent with the location of this soil resistivity midway between the high resistivity and the second most anodic (electro-negative) P/S potential measurement. Measured P/S ranged from a most cathodic -0.2628V at Station 5+60 to a most anodic value of -0.599V at Station 26+88. This entire range of P/S is within the normally expected range for buried iron. The CIS profile (Figure 7) indicates three locations that are significantly more anodic than the remainder of the force main; one centered on Station 3+57, one centered on Station 12+30, and the most anodic location at Station 26+88. The anodic indication at Station 26+88 was not recommended for direct examination because of its proximity to the end of the line and the possibility that the P/S measurements there might be influenced by connected structures. Based upon field conditions, an excavation and direct examination was conducted at Station 3+05.

The excavation at Station 3+05 found pipeline backfill composed primarily of sand. The excavation was dry and there was no evidence of migration of native soil into the backfill. This suggests that there is little groundwater flow through this area. Soil resistivity measured in the trench was high (16000 Ω-cm), similar to that measured at grade. Condition of the exposed pipe was nearly pristine, with no evidence of corrosion. This is consistent with the high soil resistivity and dry conditions.

No corrosion was found during the direct examination at Station 3+05 making it impossible to extrapolate conditions to the remainder of the pipeline. One possibility is that while this section of the force main has a high natural potential for corrosion, environmental conditions inhibit the corrosion reaction. Under this scenario, only the two areas (Station 12+30 & Station 26+88) with more anodic P/S potentials are likely to be corroding, although we cannot estimate at what rate; and the remainder of the pipeline should be essentially corrosion free. Alternatively, the corrosion causing this anodic indication may be located somewhat up-station from the area directly examined and the excavation may simply have missed it. If this is the case we can make no statement regarding the level of corrosion on the force main.

Because the excavated pipe was in such excellent condition and the coating was intact, no anode was installed at this location. The twenty-seven year history with no corrosion at Station 3+05 suggests that no corrosion failures are likely at this location in the foreseeable future, however this cannot be assumed for the remainder of the force main. This leaves two alternatives. First, a more detailed study could be performed to assess the condition of the remainder of the pipeline. Alternatively, since this force main is electrically continuous, it could easily be placed under cathodic protection preventing any further corrosion. The latter is probably the most cost-effective option.

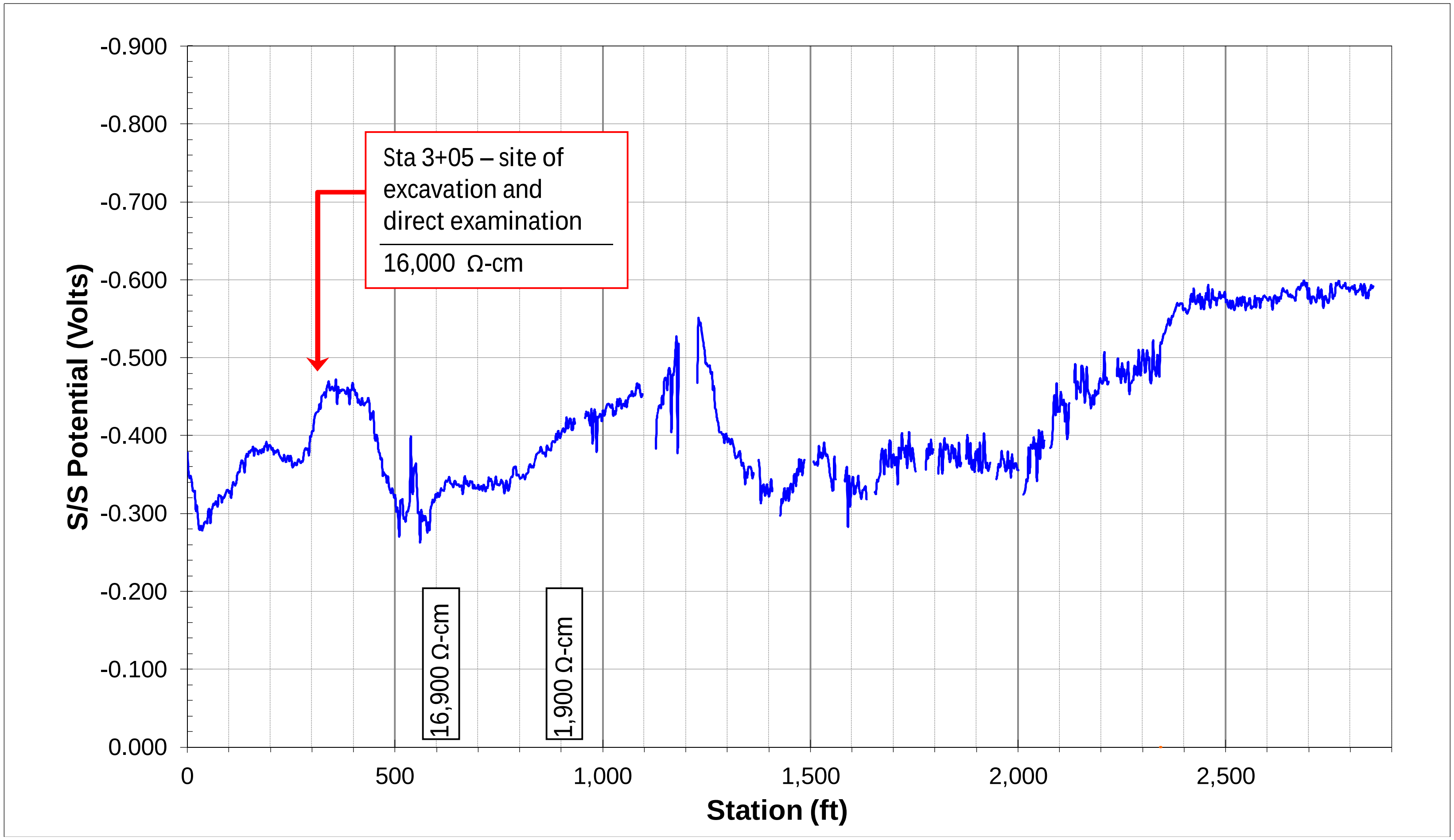


Figure 7 – Greenmeadow Force Main CIS Profile

Figure 8 – Greenmeadow Excavation Site

2.5 HEYER DRIVE

Testing of the Heyer Drive force main was performed from Station 3+60 to Station 22+80, utilizing the two-cell technique. Soil resistivity was measured at three locations, Station 1+60, Station 12+30, and Station 22+30 with readings of 10,500 Ω-cm, 3,300 Ω-cm, and 2,900 Ω-cm respectively. Seventeen anode/cathode pairs are evident (see Figure 9). The majority of these are unexpectedly located on the portion of the force main having higher soil resistivity readings. Eleven of these sites were selected as potential direct examination sites; four as primary sites and the remaining seven as secondary sites.

One site, at Station 18+90, was excavated for direct examination. The measured potential reversal at this location was from +0.0103V to -0.0137V. Backfill conditions here were similar to those found on Burr Oak; a mixture of sand, gravel and clay, and significant ground moisture. Measured soil resistivity at pipe depth was slightly less than at grade (2500 Ω-cm vs. 2900 Ω-cm). There was a thick scale on the pipeline which may be providing some protection. Where the scale was removed some minor pitting (0.01" depth) was noted. As with Burr Oak, this is minor corrosion for a forty-two year old pipeline.

All seventeen anodic indications were evaluated and compared to the directly examined site (Figure 10). The extrapolation calculations suggest three locations of possible concern with potential pitting as much as fifty percent of the pipe wall thickness. Again it must be remembered that these are extrapolations from a very limited data set and should not be viewed in the same light as actual measured values. The locations of concern are at Stations 7+05, 12+00 and 18+35 (Figure 11) where thinning of the pipe wall is likely at its maximum. These areas are estimated to have pitting rates of 3 to 5 mils/yr. Consistent with the estimated maximum depth of penetration and pipe age, the estimated maximum pitting should be expected to take another forty years for pipe penetration to occur. Leakage would be expected sooner than that as internal pressure exceeds the strength of the remaining material

An anode was installed on the exposed pipeline to protect this pipe stick from corrosion. Waukesha might want to closely monitor the areas identified as having a probability of significant corrosion pitting by re-testing every 5-10 years. Since the observed corrosion has consisted of scattered pitting, the probable mode of failure, should it occur, would likely be a leak of some size rather than a sudden catastrophic failure. If opportunities to expose the force main in these areas present themselves, it should be examined for the presence of corrosion and the condition analysis adjusted to reflect any new measurements.

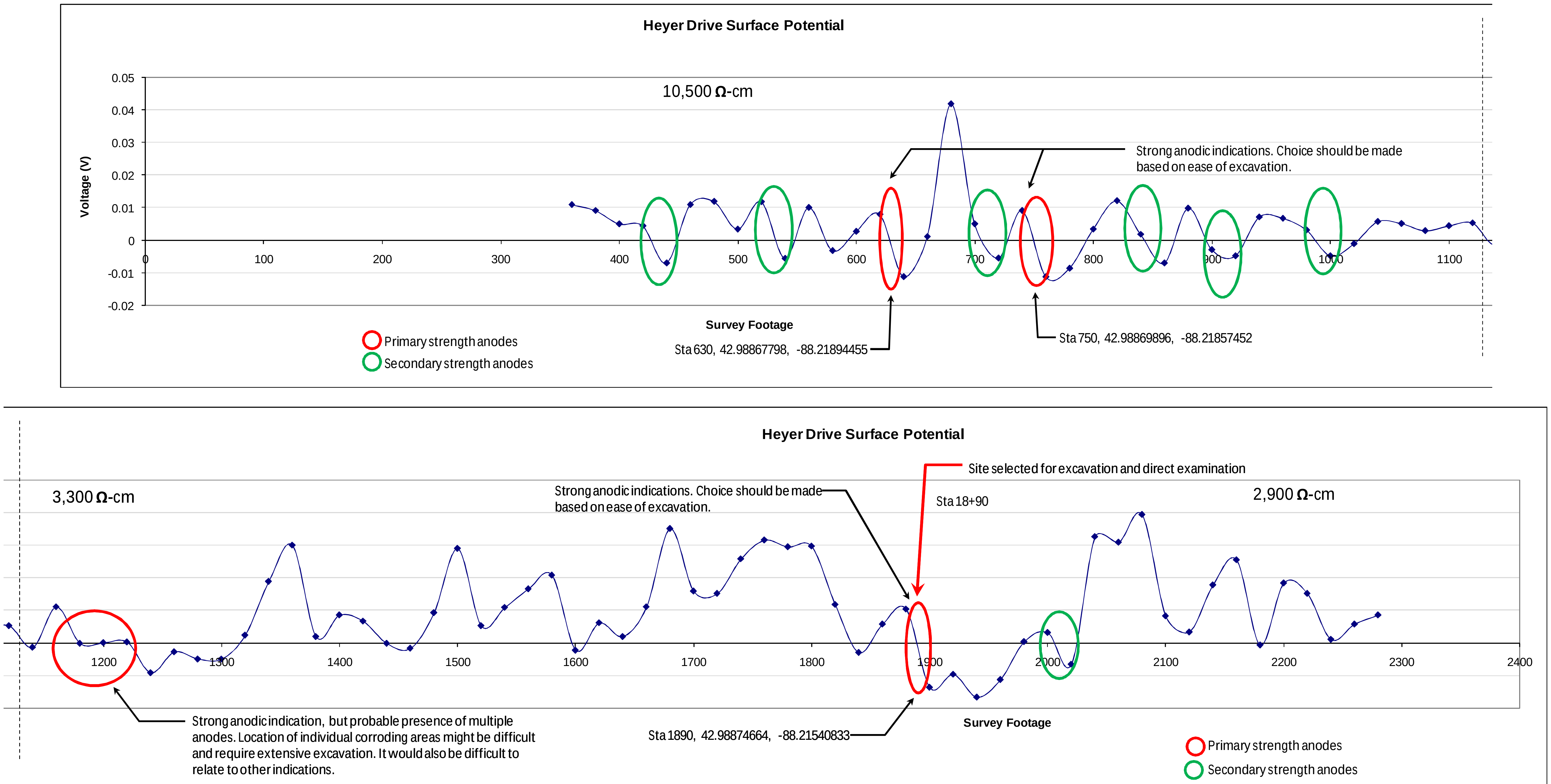


Figure 9 – Heyer Drive Two-Cell Potential Survey

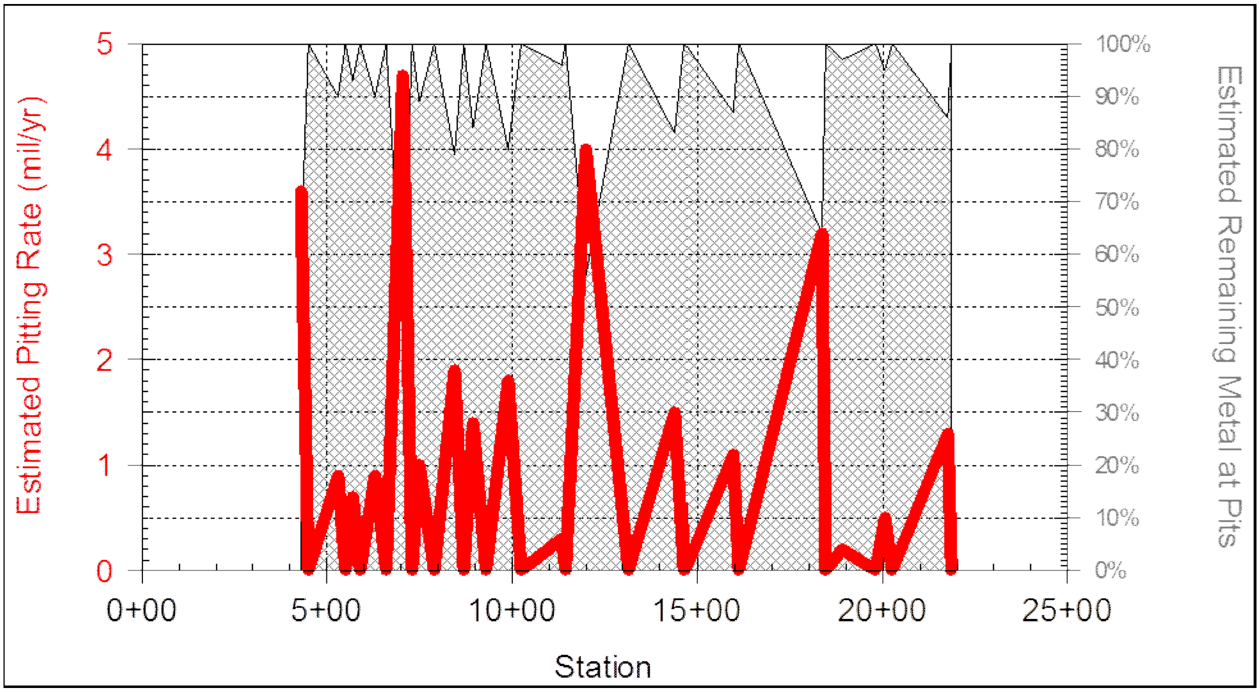


Figure 10 – Heyer Drive Force Main Estimated Condition

Figure 11 – Heyer Drive Potential Pipe Thinning Sites

2.6 PEBBLE VALLEY

The Pebble Valley force main was tested utilizing the two-cell potential profile technique from Station 0+00 to Station 43+60 (Figure 12). Soil resistivity was measured at each end of the force main, at Stations 1+30 and 41+35, with resistance of 3600 Ω-cm and 11500 Ω-cm respectively. Two-cell potential measurements ranged between -0.0597V and +0.0578V and thirty-one anode/cathode pairs were noted on the Two-cell Potential Profile. Fourteen sites were selected as candidates for direct examination with five primary sites and ten secondary sites.

Three excavations and direct examinations were performed on the Pebble Valley force main at Stations 13+00, 27+80 and 32+50. Backfill around the pipeline consisted of crushed stone. As with the other sites, a protective scale covered the pipe surface. Groundwater was encountered at Stations 13+00 and 27+80, while Station 32+50 was dry. Soil resistivity at pipe depth cannot be directly compared to that measured at grade since the excavations were not adjacent to the locations where soil resistivity was measured at the surface. Pipe depth soil resistance was measured at 2250 Ω-cm, 1500 Ω-cm, and 1800 Ω-cm. These are generally considered corrosive conditions. With one exception, the observed corrosion consisted of shallow scattered pitting. At Station 13+00 a plastic storm sewer had been laid in direct contact with the crown of the force main with slightly more concentrated pitting at the point of contact.

Figure 13 graphically depicts an evaluation of the thirty-one anodic indications. Although there were numerous indications on this force main, they do not appear to be severe. In only one case does the estimated pitting rate exceed 2 mils/yr., and at only five locations does the estimated maximum pitting depth exceed ten percent of the pipe wall thickness. The evaluation does not indicate any areas of concern on this force main.

Galvanic anodes were installed at each excavation location. These should provide local protection from corrosion. Of particular concern is the situation at Station 13+00. This emphasizes the importance of preventing contact between crossing pipes during construction.

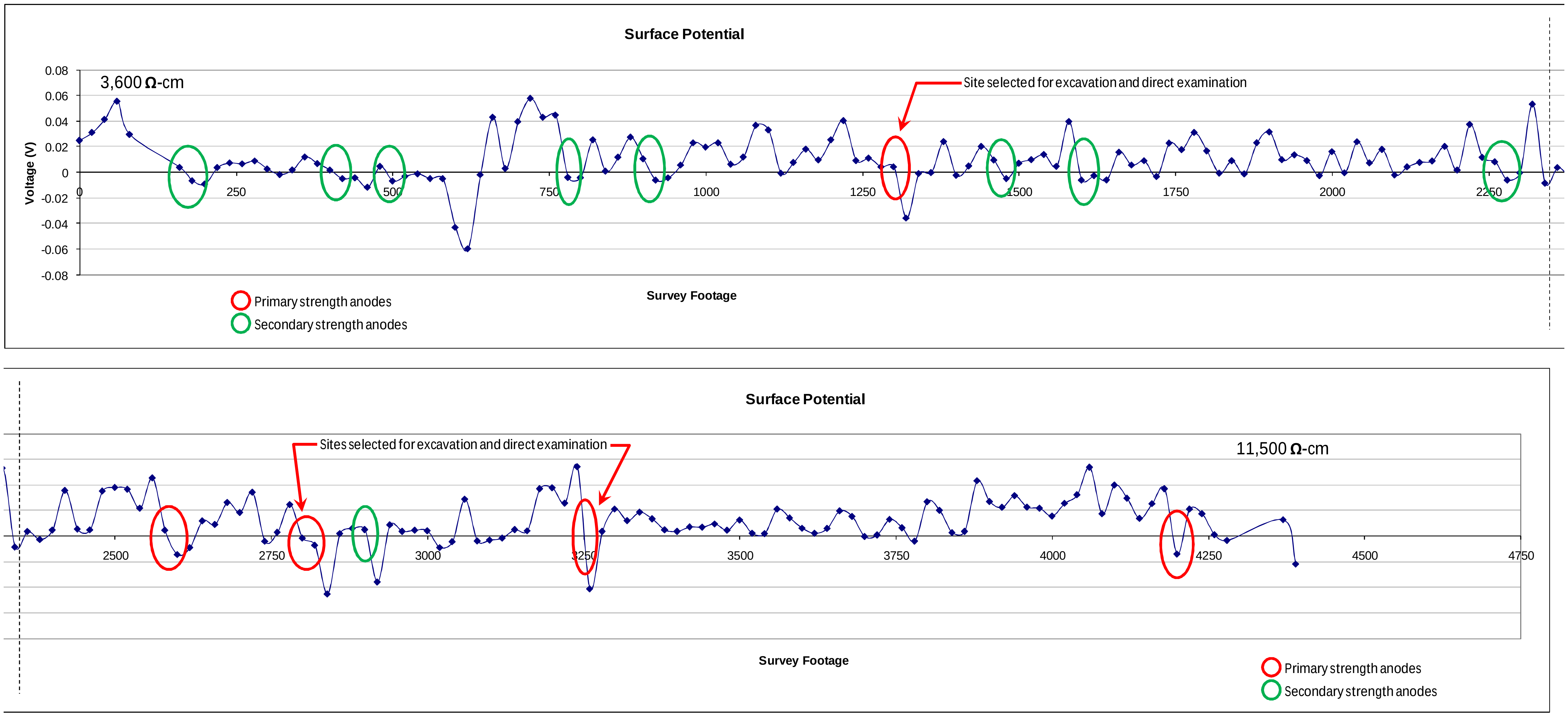


Figure 12 – Pebble Valley Two-Cell Potential Survey

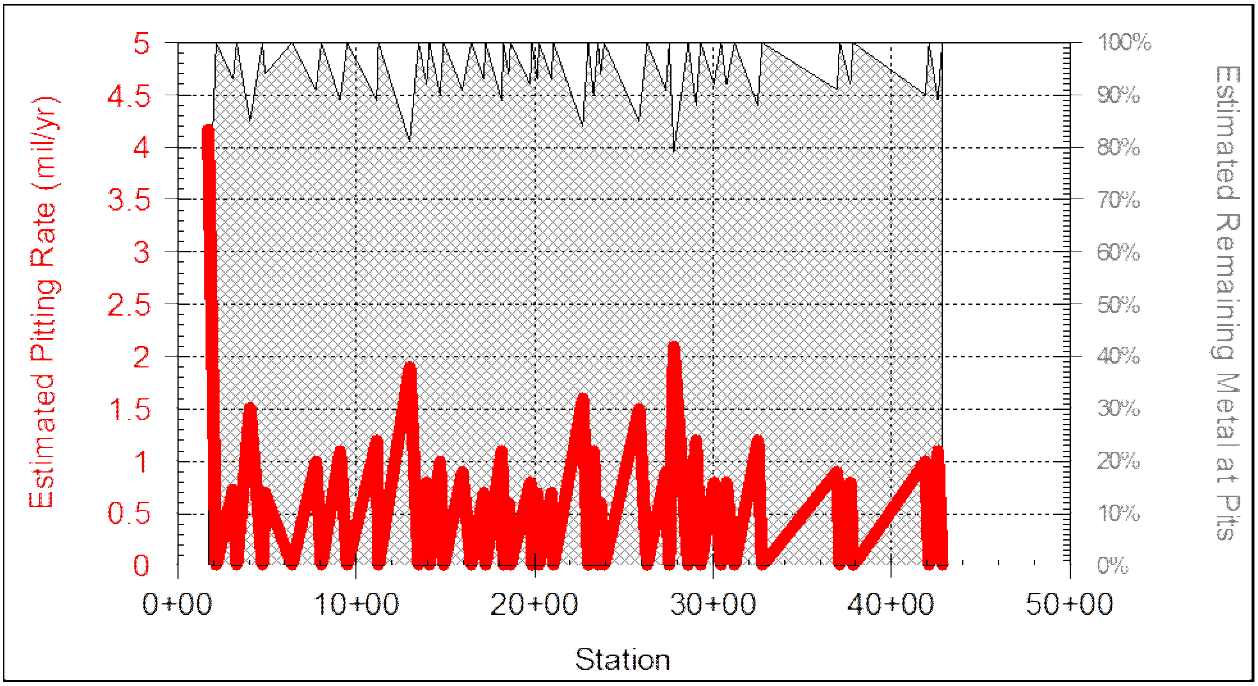


Figure 13 – Pebble Valley Force Main Estimated Condition

2.7 Conclusions

Overall, the direct examinations suggest that these highest risk force mains are in excellent condition regarding external corrosion. Several areas of greater concern have been identified and should be monitored by re-testing every 5-10 years. Bear in mind that the values presented in the assessment are extrapolations from a limited data set and should not be considered as actual levels of corrosion, rather as an indication of the approximate location and relative levels of concern for external corrosion, the principal cause of previous force main failures.

Due to the circumstances of the direct examination of the Greenmeadow force main, little can be said definitively about the condition of this pipeline. Two options to mitigate this situation are presented—more comprehensive direct examination or the installation of cathodic protection. Donohue recommends that cathodic protection be installed as the most cost-effective option.

There is no immediate need for further force main inspections. However, if opportunities to expose these force mains occur, for example during road reconstruction, they might be examined for corrosion. The results of those examinations should be incorporated into the analyses completed as part of this study, and the condition assessment modified accordingly.

CHAPTER III – HEYER DRIVE FLOW MONITORING

2008 pump run times indicated that the Heyer Drive service area experienced the highest base infiltration rates in Waukesha. To verify the accuracy of these results, an ISCO Pump Station Monitor was installed in the station in 2009. This monitor indicated that infiltration from 2008 to 2009 actually increased. To better locate where in the pump station's service area the infiltration was coming from, a supplemental flow monitoring program was implemented in 2010 by installing four flow meters upstream of the station.

The results of this analysis are shown in Figure 14, which indicates that the majority of the infiltration is originating in the area north of the pump station, and upstream of manhole 3140. This is consistent with observations made by City personnel, who have observed high clear water flow rates in the vicinity of where the Grey Terrace pump station had been.

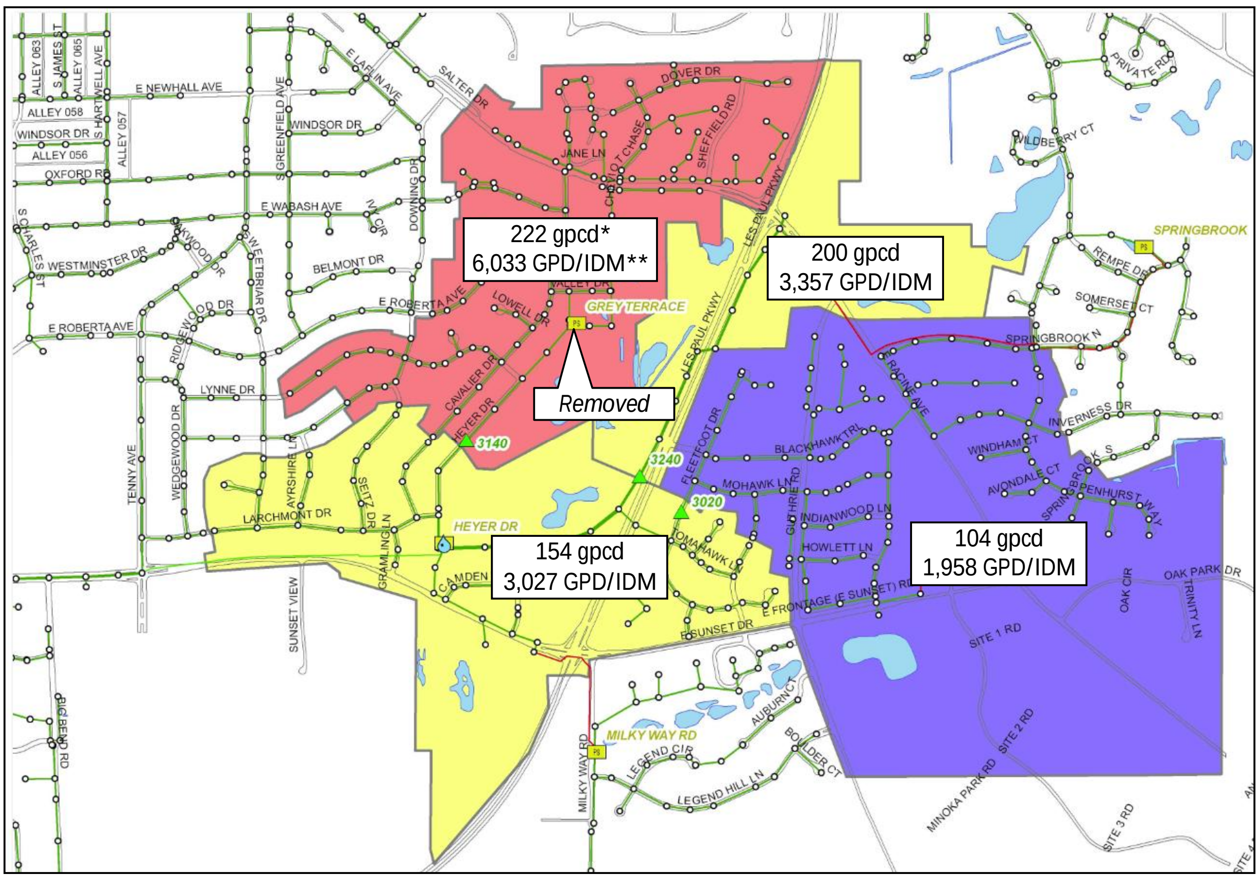


Figure 14 – Heyer Drive Infiltration

* gpcd: Gallons per capita per day.

** GPD/IDM: Gallons per day per inch*diameter*mile of sewer. Values greater than 3000 are generally considered excessive

CHAPTER IV –2009-2010 SMOKE TESTING

4.1 BACKGROUND

This chapter summarizes the findings of the smoke testing completed in 2009 and 2010 by Visu-Sewer.

In 2009, the sewers selected for testing were generally those where flow monitoring recorded rapid and significant increases in flow in response to rainfall. It was not possible to monitor downtown flows for this sort of characteristic response; however flow mass balances indicated that this area experiences significant volumes of inflow and infiltration (I/I). Furthermore, the advanced age of the downtown sewers made them good candidates for testing. Therefore the downtown sewers, those that fall within the 2,000-acre central area for which direct flow monitoring was not possible were selected for smoke testing in 2010.

During testing, the contractor had been instructed to take photos of the interiors of all manholes that were opened during smoke testing. The contractor failed to do so, and has therefore agreed to perform full manhole inspections of the ~670 downtown manholes that were not inspected in 2010 (Chapter 5).

4.2 RESULTS AND RECOMMENDATIONS

In total, approximately 66 miles of sewers (25% of all sewers owned by the City) were tested over the two-year period and a total of 41 defects (see Table 2) were found, an average of only one defect every 1.5 miles of sewer (see Figure 15). This relatively low value indicates that there are likely few locations where stormwater can directly enter the sanitary sewer system.

The precise locations of all defects have been provided in Table 2 with location maps in Appendix B. Photos of each defect have been included in Appendix C.

Many of the located defects were simply broken or missing clean-out caps. We recommend these be replaced. Smoke testing also revealed several system defects that warrant further inspection / testing. These defects, numbered according to the Object ID in the GIS, are discussed in Table 2, along with recommendations for further investigation / repairs.

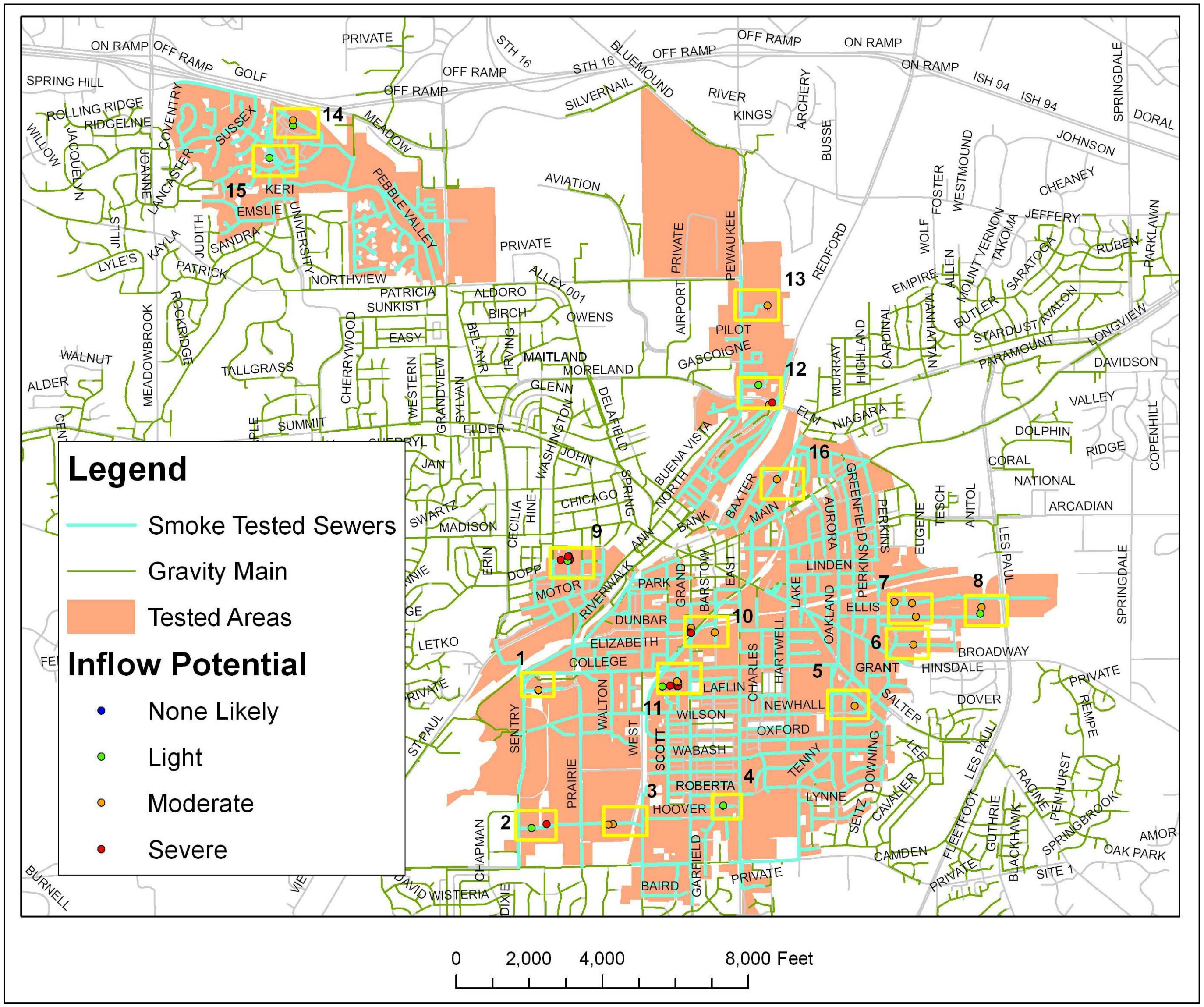


Figure 15 – Smoke Testing Areas

Table 2 – Smoke Testing Defects and Recommendations

Map #	Object ID.	Type	Severity	Location Type	Surface Cover	Location	Facility ID	Year Located	Description	X-Coor.*	Y-Coor.*	Recommendations
1	5	Storm Sewer Curb Inlet Connection	Moderate	Driveway	Paved	832 Philip Avenue	1099	2010	-	2468831	369944	Inspect and/or rehabilitate.
1	6	Mulitple Leaks	Moderate	Yard	Unpaved	832 Philip Avenue	1850	2010	-	2468832	369957	Inspect and/or rehabilitate.
2	9	Broken Cleanout	Severe	Yard	Unpaved	818 Progress Avenue	NA	2010	-	2469058	366289	
2	10	Manhole Frame/Cover	Light	Street	Paved	831 Progress Avenue	2618	2010	-	2468646	366182	Inspect and/or rehabilitate.
3	11[a]	Manhole Frame/Cover	Moderate	Yard	Unpaved	525 Progress Avenue	NA	2010	-	2470874	366284	Visually inspect if inlets are connected to sanitary sewer; if yes, reconnect inlets to storm sewer. Otherwise, dye-water flood the storm sewer whilte televising the
3	12[a]	Manhole Frame/Cover	Moderate	Yard	Unpaved	525 Progress Avenue	NA	2010	-	2470736	366277	Visually inspect if inlets are connected to sanitary sewer; if yes, reconnect inlets to storm sewer. Otherwise, dye-water flood the storm sewer whilte televising the
4	8	Manhole Frame/Cover	Light	Street	Paved	137 Coolidge Avenue	497	2010	-	2473883	366786	Inspect and/or rehabilitate.
5	97	Manhole	Moderate	Street	Paved	MH 3265	3265	2009	Cracks around MH 3265	2477474	369527	Inspect and/or rehabilitate.
6	96	Manhole	Moderate	Street	Paved	MH 3421	3421	2009	Crack around MH 3421	2479083	371200	Inspect and/or rehabilitate.
7	93	Manhole	Moderate	Street	Paved	MH 3428	3428	2009	Cracks around MH 3428	2479053	372330	Inspect and/or rehabilitate.
7	94	Clean Out	Moderate	Street	Paved	1225 Pearl St.	NA	2009	Clean Out near driveway is missing the cover	2478564	372367	Replace clean out cover.
7	95	Manhole	Moderate	Street	Paved	MH 3425	3425	2009	Crack around MH 3425	2479157	371968	Inspect and/or rehabilitate.
8	91	Manhole	Moderate	Street	Paved	MH 90	90	2009	CRACKS AROUND MANHOLE COVER #90	2480955	372224	Inspect and/or rehabilitate.
8	92	Clean Out	Light	Yard	Unpaved	400 Commerce St.	NA	2009	Clean Out missing cover	2480913	372047	Replace clean out cover.
9	105	Other	0	NA	NA	MH 2376	2376	2009	Cannot access MH inside hospital	2470271	373498	
9	107[b]	Catch Basin	Severe	Street	Paved	American Ave & Washington Ave.	1535	2009	CB 1535	2469440	373522	Dye-water flood the storm sewers while televising the sanitary sewers
9	109[b]	Catch Basin	Severe	Street	Paved	CB 28778	NA	2009	-	2469670	373525	Dye-water flood the storm sewers while televising the sanitary sewers
9	110[b]	Catch Basin	Severe	Street	Paved	CB 28779	0	2009	-	2469686	373556	Dye-water flood the storm sewers while televising the sanitary sewers
9	111[b]	Catch Basin	Severe	Street	Paved	CB 1534	1534	2009	-	2469689	373524	Dye-water flood the storm sewers while televising the sanitary sewers
9	113[b]	Ground	Moderate	Street	Paved	Precise location unknown. See photos.	NA	2009	Crack in curb leaking smoke.	2469634	373499	Dye-water flood the storm sewers while televising the sanitary sewers
9	114[b]	Ground	Light	Street	Paved	Precise location uknown. See photos.	NA	2009	Ground leaking smoke.	2469623	373499	Dye-water flood the storm sewers while televising the sanitary sewers
9	115[b]	Catch Basin	Severe	Street	Paved	Greenwood Ave Between Lawndale and American	NA	2009	-	2469665	373627	Dye-water flood the storm sewers while televising the sanitary sewers
9	116[b]	Catch Basin	Severe	Street	Paved	Greenwood Ave Between Lawndale and American	NA	2009	-	2469636	373615	Dye-water flood the storm sewers while televising the sanitary sewers
10	1[c]	Storm Sewer Curb Inlet Connection	Moderate	Street	Paved	240 Barstow Street	105	2010	-	2473651	371531	Dye-water flood the storm sewers while televising the sanitary sewers
10	2[c]	Storm Sewer Curb Inlet Connection	Moderate	Street	Paved	307 Grand Avenue	2114	2010	-	2473011	371659	Dye-water flood the storm sewers while televising the sanitary sewers
10	3[c]	Other, See Comments	None Likely	Street	Paved	Grand Avenue and Williams Street	9475	2010	-	2472972	371536	Dye-water flood the storm sewers while televising the sanitary sewers
10	4[c]	Storm Sewer Curb Inlet Connection	Severe	Street	Paved	301 Grand Avenue	2113	2010	-	2473010	371523	Check if inlet is directly connected to the sanitary sewer. Dye-water flood the storm sewer while televising the sanitary sewer.
11	13[d]	Storm Sewer Curb Inlet Connection	Severe	Street	Paved	204 Hinman Avenue	NA	2010	-	2472623	370060	Dye-water flood the storm sewer while televising the sanitary.
11	14[d]	Storm Sewer Curb Inlet Connection	Severe	Street	Paved	201 Hinman Avenue	NA	2010	-	2472652	370061	Dye-water flood the storm sewer while televising the sanitary.
11	15[d]	Storm Sewer Curb Inlet Connection	Moderate	Street	Paved	131 Hinman Avenue	4444	2010	-	2472652	370167	Dye-water flood the storm sewer while televising the sanitary.
11	16[d]	Storm Sewer Curb Inlet Connection	Moderate	Street	Paved	132 Hinman Avenue	NA	2010	-	2472622	370192	Dye-water flood the storm sewer while televising the sanitary.
11	17[d]	Storm Sewer Direct Connection	Severe	Street	Paved	345 Harvey Avenue	1250	2010	-	2472435	370081	Dye-water flood the storm sewer while televising the sanitary.
11	18	Broken Cleanout	Light	Yard	Unpaved	411 Harvey Avenue	NA	2010	-	2472225	370048	Inspect and/or rehabilitate.
12	99	Clean Out	Light	Yard	Paved	314 Morey St.	NA	2009	Clean Out cover is cracked	2474843	378329	Inspect and/or rehabilitate.
12	100	Manhole	Moderate	Driveway	Paved	MH 3787 (1418 E North St.)	3787	2009	Crack around Manhole indriveway.	2475146	377781	Inspect and/or rehabilitate.
12	101	Catch Basin	Severe	Street	Paved	MH 3788	3788	2009	Storm grate next to manhole is leaking smoke	2475228	377840	Inspect and/or rehabilitate.
13	108	Manhole	Moderate	Sidewalk	Paved	MH 3093	3093	2009	LEAKS BETWEEN CONCRETE SLAB AND THE STONE	2475091	380498	Inspect and/or rehabilitate.
14	84	Clean Out	Light	Yard	Unpaved	2912 Fielding Lane	NA	2009	CLEAN OUT COVER IS CRACKED	2462119	385451	Replace clean out cover.
14	85	Manhole	Moderate	Driveway	Paved	MH 5613	5613	2009	SMOKE ESCAPED FROM CRACK AROUND MANHOLE	2462117	385587	Inspect and/or rehabilitate.
15	86	Clean Out	Light	Yard	Unpaved	2712 University Ct	NA	2009	CRACKED CLEAN OUT COVER IN FRONT OF HOUSE.	2461468	384537	Replace clean out cover.
16	7	Broken Cleanout	Moderate	Yard	Unpaved	1165 White Rock Avenue	NA	2010	-	2475350	375737	

* State-Plane Coordinates

a – Defects 11 & 12 are yard drains that were smoking heavily during testing. While these inlets do not appear in GIS records, the GIS indicates there is a storm sewer directly adjacent to these inlets. However, none of the other inlets along this storm sewer were smoking. These inlets are most likely directly connected to the sanitary sewer. We recommend Waukesha perform a visual inspection to confirm whether this is the case, and if so, reconnect these inlets to the storm sewer. If not, we recommend Waukesha dye-water flood the storm sewer while televising the sanitary sewer.

b – There is a cluster of inlet defects (107, 109-111, 115, 116) and non-inlet defects (113 & 114) near the hospital (map Page 9 in Appendix B). None were smoking heavily. These are likely the result of both the sanitary and storm sewers being in poor structural condition where the two systems cross. We recommend dyed-water flooding the storm sewers while televising the sanitary sewers.

c – Defect 4 (map page 10 in Appendix B) is an inlet that was smoking heavily and is nearest where the storm sewer crosses over the sanitary. Defect 3 is a crack in the pavement adjacent to a sanitary manhole. Defect 2 is an inlet located 135' to the north on the storm line. We recommend checking whether Defect 4 is an inlet directly connected to the sanitary sewer (unlikely). More likely are structural defects in both the sanitary and storm sewer systems, particularly where they cross near the intersection of Williams and Grand. We recommend dye-water flooding the storm sewer while televising the sanitary sewer to confirm the existence and severity of I/I from the storm sewer into the sanitary via structural defects. Defect 1 is an inlet that was not smoking heavily. This is likely due to structural defects where the sanitary and storm sewers cross. We recommend dyed-water flooding the storm sewer while televising the sanitary.

d – Defects 13-16 (map Page 11 in Appendix B) are four inlets along Hinman that appear to have been installed when the street was reconstructed. They were all smoking heavily. Defect 17 is a storm manhole that was also smoking heavily. There appears to be a cross connection between the sanitary and storm systems near the intersection of Hinman & Harvey. We recommend checking record drawing for the possible location of the cross connection. Dyed-water flooding of the storm sewer while televising the sanitary may be required to locate and repair the defect.

CHAPTER V – 2010 MANHOLE INSPECTIONS

5.1 BACKGROUND

In the summer of 2010, as part of Waukesha's Sanitary Sewer Evaluation Survey (SSES), the City chose to inspect approximately 500 manholes along some of the oldest portions of the collection system, predominantly in the downtown area. These manholes represent approximately 7% of the total sanitary manholes owned by the City. Visu-Sewer conducted the manhole inspections. They took digital photographs and noted the condition of the interior and the surrounding surface of each inspected manhole.

5.2 METHODOLOGY

In all, Visu-Sewer completed 477 manhole inspections. They also inspected the majority of the manholes on the City's "30-day list", a record of 115 manholes that require frequent cleaning due to various conditions such as structural defects, grease, or sewer sags. Visu-Sewer populated a database containing the fields shown in Table 3 on the following page.

Donohue used this database to develop a rating system, categorizing the manholes as being in excellent, good, fair, or poor condition. Of the 477 manholes inspected, 368 (77%) were found to be in good to excellent condition, as shown in Figure 16. The locations of these manholes are shown in Figure 17. These ratings are based on a generic rating algorithm; therefore, Donohue evaluated the specific defects of each manhole in order to develop specific rehabilitation recommendations.

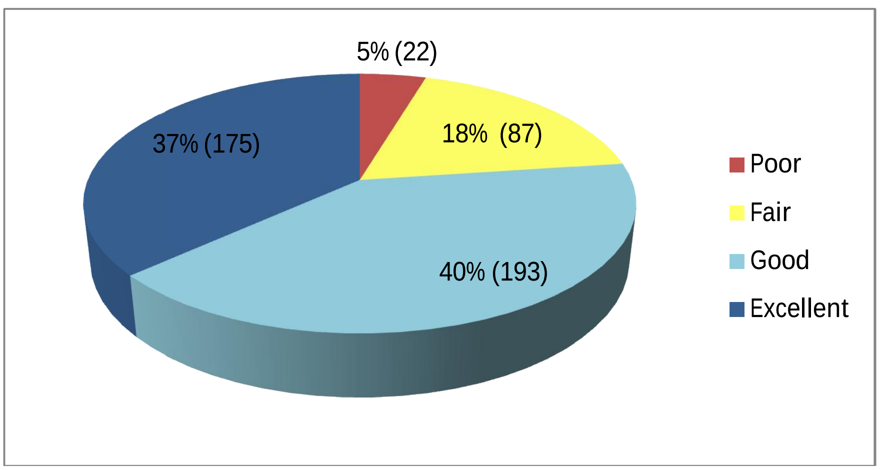


Figure 16 – Condition of Inspected Manholes

Table 3 – Data Dictionary for 2010 Manhole Inspections

Condition/Characteristics		
Facility ID	**Chimney Material**	**Bench Condition**
Date	Brick	Good
Inspector	Concrete rings	Fair
Basin ID	Plastic/Rubber Rings	Aggregate/Rough Concrete
Street	**Chimney Seal Condition**	No bench / no lateral through
Ponding Potential	Good	See Comments
Low	Leaking	**Material on Bench**
Moderate	None	Yes
High	**Chimney Defects**	No
Surface Cover	Cracked	**Invert Condition**
Asphalt	Missing Brick	Good
Concrete	Loose Mortar	Fair
Gravel	Other	Poor
Grass	**Chimney Height (inches)**	**Invert Sediment/Debris**
Surface Cracking	**Cone Material**	Minor
Minor	Brick	Moderate
Fair	Block	Significant
Severe	Monolithic	**Steps Condition**
Frame Condition	Precast Concrete	Good
Good	**Cone Defects**	Fair
Cracked	Cracked	Missing
Worn	Missing Brick	None
New Cover	Loose Mortar	**Evidence of Surcharge**
Yes	Other	Yes
No	**Barrel Material**	No
Cover Condition	Brick	**Infiltration**
Good	Block	Stream
Cracked	Monolithic	Sheet
Worn	Precast Concrete	Wet
Cover Hole Count (number)	**Barrel Defects**	Mineral Deposits
Cover Gasket Condition	Cracked	**Comments**
Good	Bad Joint	**Leaking**
Poor	Missing Brick	
None	Loose Mortar	
Frame Offset (inches)		
Leaking		

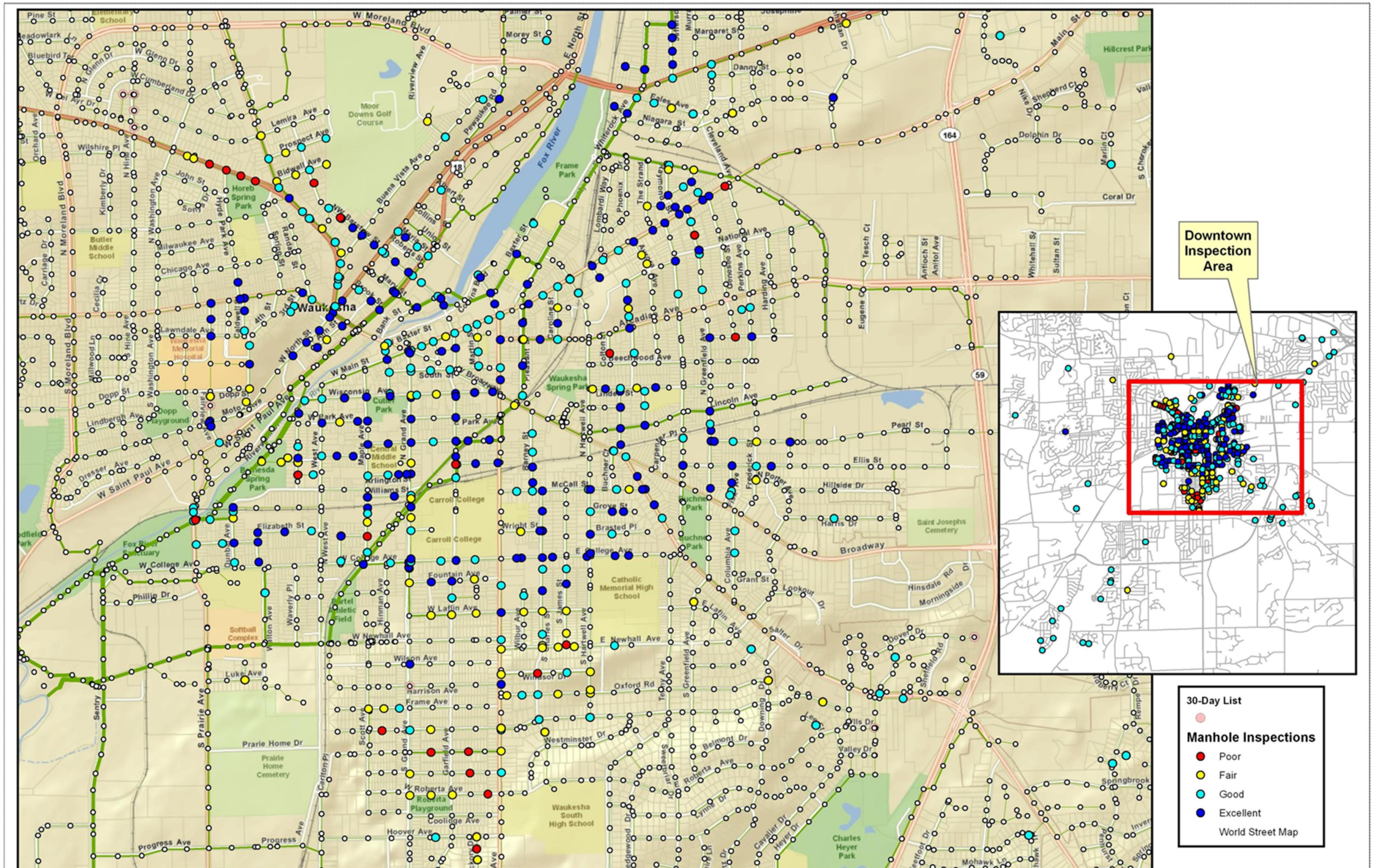


Figure 17 – Location and Condition of Inspected Manholes

Rehabilitation and replacement recommendations were based on data received from manhole inspection data. This data was compiled into an Access database and cross-referenced with the 30-day cleaning list, street reconstruction projects, and manhole characteristics. To facilitate the review, an inspection form (Figure 18) was created to view the characteristics, defects, and pictures of each manhole. Reviewers populated the fields near the top and bottom of the form, recommending specific rehabilitation methods and priorities appropriate for the manhole condition information from the middle of the form.

Figure 18 – Manhole Inspection Form

Rehabilitation and replacement recommendations were made using the following categories:

- Rehabilitate – some components require rehabilitation
- Replace – the entire manhole should be replaced
- Partial Replace – only some components of the manhole require replacement
- Re-Inspect – the manhole should be re-inspected before replacement or rehabilitation can be recommended
- None – take no action; the manhole is in good or excellent condition

Manholes were prioritized using the following categories:

- High – given to manholes that seem to be in danger of collapsing
- Moderate – given to all manholes on the "30-day list", as well as manholes that have severe problems
- With street reconstruction – given to manholes located in areas scheduled for street reconstruction
- Re-inspect in 5 years – given to manholes that have some problems, but do not warrant rehab or rehabilitation at this time
- Low – given to all other manholes that require rehabilitation or replacement.

Donohue used the recommendations to prepare cost estimates for rehabilitating or replacing each structure. Recommendations and costs were prepared using the following methodology:

- Repair Pavement
  Repairing the pavement was recommended if surface cracking was severe, which can be a cause of infiltration. In some cases, field crews did not note severe cracking but the accompanying picture showed excessive cracks; in these cases, repairing the pavement was also recommended. Costs for pavement repair were not included in the cost estimates, however.

- New Cover
  If pick holes were present in the cover, replacement was recommended. Since the City already owns covers, a cost for this was only included if no other rehab or replacement was recommended. A unit price of $50 was used for installation.

- Replace Frame
  We recommended replacing the frame if the inspection report indicated it was cracked or worn. Per the City's comments, frame replacements were recommended in conjunction with chimney replacements – if the frame needs replacement, often the chimney does also. Costs for this replacement were recommended by the City to be $350 per vertical foot with a one-foot minimum.

- Rehab Barrel
  Barrel rehabilitation was recommended if the inspection report indicated there is a bad joint, loose mortar, or the barrel is cracked or leaking. For manholes with brick barrels, a complete replacement was recommended unless the inspection report indicated no defects. For rehabilitation, we assumed a cementitious liner would be used with a price of $125 per vertical foot, or approximately $1,200 for the entire structure. Since exact barrel dimensions are unknown, the $1,200 figure was used for any barrel rehabilitation.

- Rehab Bench / Invert / Trough
  Rehabilitation was recommended for benches and inverts if the inspection reports indicated they were in poor condition. Several reports made note of lateral lines that had no trough to direct flow. In those cases, we recommended building a trough for those lines.

  For several manholes, the inspection report did not indicate any defects, but the pictures or notes from cleaning crews were not in agreement. In those cases, rehabilitation was recommended with a “Field Verify” note.

  The cost for rehabilitation was estimated at $70/hr for a 2-person crew, assuming the crew would need 4 hours to complete the work. Where a trough or invert rehabilitation was needed, a cost of $100 per manhole was included for bypass pumping.

- Rehab Cone/Chimney
  Rehabilitation was recommended for cones and chimneys when inspection reports indicated they were cracked or there was loose mortar. However, if the chimney or cone was brick, a complete replacement was recommended. For rehabilitation, we assumed a cementitious liner would be used with a price of $125 per vertical foot. Chimney height was known for every manhole; 4 feet was assumed to be the height of each cone.

- Leaking Manhole
  In instances where inspection crews noted a leaking manhole, grouting was recommended based on the City’s preferences. An average cost of $450 per manhole was used for this repair.

- Total Manhole Replacement
  Manholes with severe defects were recommended for complete replacement. Most of these manholes are made of brick and are not worth rehabilitating. An average cost of $450 per vertical foot was used. Manhole depth was calculated by subtracting the elevation of the invert from the rim elevation. This data was available for approximately half of the manholes inspected; average depth for these manholes is 14.5 feet. For all manholes without elevation data, a depth of 14.5 feet was used.

5.3 RECOMMENDATIONS

Table 4 summarizes the results of the inspections and costs of manhole rehabilitation by priority.

Table 4 – Manhole Rehabilitation Priority

Recommendation → / ↓ Priority	None		Partial Replace		Rehab		Re-inspect		Replace		
	Cost	Total Number	Cost	Total Number	Cost	Total Number	Cost	Total Number	Cost	Total Number	Total Cost
-	$ -	167	$ -	-	$ -	-	$ -	-	$ -	-	$ -
High	$ -	-	$ -	-	$ 1,404	1	$ -	-	$ -	-	$ 1,404
Low	$ 6,650	-	$ 24,050	38	$ 82,445	126	$ 1,460	1	$ 3,785	1	$118,389
Moderate	$ -	-	$ 10,919	7	$ 79,135	78	$ 660	7	$ 52,920	13	$143,634
Re-inspect in 5 years	$ -	-	$ -	-	$ -	-	$ -	5	$ -	-	$ -
With street reconstruction	$ -	-	$ 5,222	3	$ 22,179	28	$ -	-	$ 4,500	1	$ 31,901
High/With street reconstruction	$ -	-	$ -	-	$ -	-	$ -	-	$ 4,500	1	$ 4,500
Totals	$ 6,650	167	$ 40,190	48	$185,163	233	$ 2,120	13	$ 65,705	16	$299,827

Detailed manhole condition assessments and rehabilitation recommendations for each of the 477 inspected manholes have been included in Appendices D and E. Full reports that include digital photographs can be printed from the inspection database provided electronically. This information should be of use to construction contractors hired to perform manhole rehabilitation.

5.4 Future Work

For future inspections, we recommend that field crews utilize the new NASSCO manhole inspection database. This standardized and more robust format will enable engineers to make rehabilitation recommendations with less reliance on their interpretation of photographs. This database has been delivered to the City electronically. Paper versions of the inspection form and associated lookup tables have been included (Table 5 and Table 6). Waukesha is in the process of incorporating an inspection form into their VueWorks Asset Management system.

Table 5 – NASSCO Manhole Inspection Form

Box 1 – MANHOLE INSPECTION DATA

Manhole ID:	Owner: City of Waukesha	Weather:	Date:
Inspector:	Certificate #:	Location Code:	Time:

Box 2

Status:	Runoff Potential: ❒ Inundated ❒ None ❒ Ponding ❒ Sheeting	Surface Cracking: ❒ None ❒ Minor ❒ Fair ❒ Severe	Steps Condition: ❒ Good ❒ Fair ❒ Poor ❒ Missing ❒ None
MH Use:			
Access Type:	Purpose:	Cover Shape: ❒ Circular ❒ Oval ❒ Rectangular ❒ Square	
# Holes:	Surface:		
❒ New Cover	❒ Evidence of Surcharge		Cover Size: ________ inches

Box 3 – MANHOLE CONDITION

Cover

Type: ❒ Solid ❒ Single Hatch ❒ Vented ❒ Double Hatch ❒ Gasketed ❒ Locking ❒ Bolted ❒ Lamphole

Fit: ❒ Good ❒ Mismatched to frame ❒ Undersized ❒ Rocks/Wobbles ❒ Oversized

Condition: ❒ Sound ❒ Bolts Missing ❒ Broken ❒ Restraint Defective ❒ Cracked ❒ Restraint Missing ❒ Corroded ❒ Missing

Insert

Insert Condition: ❒ Sound ❒ Leaking ❒ Poor Fit ❒ Insert Fell ❒ Cracked ❒ Corroded

Insert Type: ❒ None ❒ Metal ❒ Other ❒ Plastic

Adjustment Ring

Condition: ❒ Sound ❒ Corroded ❒ Broken ❒ Cracked ❒ Leaking ❒ Poor Install

Type: ❒ None ❒ Adjustable ❒ Solid

Material: ❒ Cast Iron ❒ Steel

Height: ________ inches

Frame

Condition: ❒ Sound ❒ Corroded ❒ Cracked ❒ Coated ❒ Broken ❒ Missing

Offset: ______ inches

Material: ❒ Cast Iron ❒ Reinforced Concrete ❒ Unknown ❒ Other

Seal Condition: ❒ Cracked ❒ Offset ❒ Sound ❒ Missing ❒ Loose

Seal Infiltration: ❒ None ❒ Dripper ❒ Gusher ❒ Runner ❒ Stained ❒ Weeper

Chimney

Material: ____________

Lining: ______________

Depth: inches

Infiltration: ❒ None ❒ Dripper ❒ Gusher ❒ Runner ❒ Stained ❒ Weeper

Barrel

Material: ______________

Lining: ______________

Diameter: _________ inches

Bench

Condition: ❒ Material on Bench ❒ No Bench ❒ Partial Bench

Material: ❒ Reinforced Concrete ❒ Non-Reinforced Concrete

Lining: ____________________

Channel

Type: ❒ None ❒ Formed ❒ Hand-Formed ❒ Insert ❒ Pipe ❒ Precast

Sediment: ❒ None ❒ Minor ❒ Moderate ❒ Significant

Exposure: ❒ Closed ❒ Fully Open ❒ Partially Open

Material: ❒ PVC ❒ Non-Reinforced Concrete ❒ Other ❒ Reinforced Concrete ❒ Vitrified Clay

Box 4 – MANHOLE DEFECTS

Component	MACP Code	Dim 1	Dim 2	Value %	Joint

Photos

Image Name 1:
Image Name 2:
Image Name 3:
Image Name 4:
Image Name 5:
Image Name 6:

Box 5 – REHABILITATION RECOMMENDATIONS

❒ Field Verify ❒ Repair Pavement ❒ Replace Cover ❒ Replace Frame ❒ Rehab Wall ❒ Rehab Bench ❒ Rehab Channel ❒ Rehab Invert

Chimney: ❒ Rehab ❒ Replace

Cone: ❒ Rehab ❒ Replace

Recommendation: ❒ None ❒ Replace ❒ Re-Inspect ❒ Partial Replace ❒ Rehab

Priority: ❒ High ❒ Moderate ❒ Low ❒ Re-Inspect in 5 years ❒ With Street Reconstruction

Comments:

Table 5 – NASSCO Manhole Inspection Form (cont)

Box 6	MANHOLE LOCATION DETAILS				
	Street:		Location Details:		
	City:				
	Coordinates:				
	x:		y:		

Box 7	PIPE CONNECTIONS						
	Pipe Number	Clock Position	Material	Seal Condition	Dia1	Dia2	Pipe Condition:
	1			❐ Defective ❐ Sound			
		Comments:					
	2			❐ Defective ❐ Sound			
		Comments:					
	3			❐ Defective ❐ Sound			
		Comments:					
	4			❐ Defective ❐ Sound			
		Comments:					
	5			❐ Defective ❐ Sound			
		Comments:					
	6			❐ Defective ❐ Sound			
		Comments:					
	7			❐ Defective ❐ Sound			
		Comments:					
	8			❐ Defective ❐ Sound			
		Comments:					

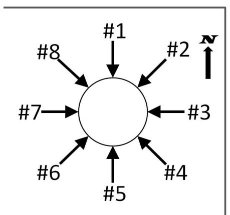


Table 6 – NASSCO Manhole Inspection Lookup Table

Box 1

Weather	Abbv.
Dry	1
Heavy Rain	2
Light Rain	3
Snow	4
Saturated	5
Damp	6
Very Dry	7
Location Code	**Abbv**
Airport	M
Alley	H
Building	J
Creek	K
Ditch	I
Easement/Right of Way	D
Light Highway	C
Main Highway - Suburban/Rural	B
Main Highway - Urban	A
Other	Z
Parking Lot	G
Railway	L
Sidewalk	F
Woods	E

Box 3

Material	Abbv.
Block	BL
Brick	BR
Cast Iron	CAS
Concrete (non-reinforced)	CN
Concrete (reinforced)	CR
Ductile Iron	DI
Fiberglass Reinforced	FR
Not Known	XXX
Other	ZZZ
Plastic/Steel Composite	PSC
Polyethylene	PE
Polyvinyl Chloride	PVC
Steel	S
Stone	ST
Vitrified Clay Pipe	VCP
Lining	**Abbv.**
Bitumastic	B
Cementitious	C
Cured in Place	CP
Epoxy	E
Fiberglass	F
None - No Coating	NC
Other	ZZ
Plastic	PL
Polymer	P
Rubber	R

Box 2

Status	Abbv.
Buried or Marked	BM
Descent Inspection	DI
No Access	NA
Not Found	NF
Not Opened	NO
Remote Inspection	RI
Surcharged/Debris	SD
---- Surface Inspection ---	- SI -
Manhole Use	**Abbv.**
Combined	CB
Force Main	FM
Other	ZZ
Processes	PR
---- Sanitary ----	- SS -
Stormwater	SW
Access Type	**Abbv.**
Catch Basin	ACB
Clean Out House	ACOH
Clean Out Mainline	ACOM
Clean Out Property	ACOP
Junction Box	AJB
---- Manhole ----	- AMH -
Meter	AM
Other Special Chamber	AOC
Wastewater Access	AWA
Wet Well	AWW
Purpose	**Abbv.**
Capital Improvement Program Assessment	G
Infiltration and Inflow investigation	B
Maintenance related	A
Not known	Z
Post rehabilitation survey	C
Pre acceptance - new sewers	E
Pre rehabilitation survey	D
Resurvey for any reason	H
Routine assessment	F
---- Sewer System Evaluation Survey ----	- I -
Surface	**Abbv.**
---- Asphalt ----	- AS -
Concrete Pavement	CP
Concrete Collar	CC
Grass/Dirt	GD
Gravel	GR

Table 6 – NASSCO Manhole Inspection Lookup Table (cont)

	Description	MACP Code	Description	MACP Code
	Crack Circumferential	CC	Roots Ball Barrel	RBB
	Crack Longitudinal	CL	Roots Ball Connection	RBC
	Crack Multiple	CM	Roots Ball Joint	RBJ
	Deformed	D	Roots Ball Lateral	RBL
	Deposits Attached Encrustation	DAE	Roots Fine Barrel	RFB
	Deposits Attached Grease	DAGS	Roots Fine Connection	RFC
	Deposits Attached Other	DAZ	Roots Fine Joint	RFJ
	Deposits Attached Ragging	DAR	Roots Fine Lateral	RFL
	Deposits Ingressed Fine	DNF	Roots Medium Barrel	RMB
	Deposits Ingressed Gravel	DNGV	Roots Medium Connection	RMC
	Deposits Ingressed Other	DNZ	Roots Medium Joint	RMJ
	Deposits Settled Compacted	DSC	Roots Medium Lateral	RML
	Deposits Settled Fine	DSF	Surface Aggregate Visible	SAV
	Deposits Settled Gravel	DSGV	Surface Missing Wall	SMW
	Deposits Settled Other	DSZ	Surface Spalling	SSS
	Displaced Brick	DB	Surface Other	SZ
	Hole	H		
	Infil Dripper	ID		
	Infil Gusher	IG		
	Infil Runner	IR		
	Infil Stain	IS		
	Infil Weeper	IW		
	Joint Offset Large	JOL		
Box 4	Joint Offset Medium	JOM		
	Joint Separated Large	JSL		
	Joint Separated Medium	JSM		
	Buckling Wall	KW		
	Lining Failure Annular Space	LFAS		
	Lining Failure Blistered	LFB		
	Lining Failure Buckled	LFBK		
	Lining Failure Bulges	LFBU		
	Lining Failure Detached	LFD		
	Lining Failure Discoloration	LFDC		
	Lining Failure Delaminating	LFDL		
	Lining Failure Pinhole	LFPH		
	Lining Failure Wrinkled	LFW		
	Lining Failure Other	LFZ		
	Missing Brick	MB		
	Mortar Missing Large	MML		
	Mortar Missing Medium	MMM		
	Mortar Missing Small	MMS		
	Repair Other Defective	RPZD		

Manhole Component	Abbv.
Bench	B
Channel	C
Chimney	CMI
Cone	COI
Wall	WI

CHAPTER VI – WEST-SIDE & SOUTHEAST INTERCEPTORS ALTERNATIVE EVALUATION

6.1 BACKGROUND

The City of Waukesha currently operates and maintains 42 pump stations that convey wastewater flows to the treatment plant. Some of these pump stations have experienced operational issues during extreme wet weather events, power failures, overflows, and flooding. Force main failures remain one of the greatest threats to the integrity of the collection system. This section summarizes the feasibility of replacing pump stations with gravity mains as topography permits.

Preliminary sizing and routing were preformed during Phase I of this Master Planning effort. In this second phase, more detailed pump station elimination analyses were performed. These analyses included evaluating additional alternatives, inflow/infiltration reduction, future flows, and preparing updated cost estimates. Ultimately, an interceptor running north to south along the west side of town and an interceptor in the southeast part of town could eliminate up to 12 pump stations with daily average and peak flows totaling 7.46 MGD and 29.2 MGD respectively (see Table 7). Please note that the southeast interceptor would not completely eliminate the need for pumping; rather, these flows will be consolidated at a new Fox Point pump station with a firm capacity of 13 MGD.

6.2 FUTURE FLOW PROJECTIONS

Waukesha currently serves an area of approximately 25 square miles. The existing system and proposed interceptors were evaluated so as to provide adequate capacity to serve the City's entire sanitary sewer service area (SSA), much of which is currently undeveloped or on septic, totaling 47 mi^2. The flows from these projected users were included in the sizing of the interceptors. Projected new users will contribute an additional 2.78 MGD of daily average flow (6.95 MGD peak) into the system. These flows were assigned to the nearest manhole that did not require crossing a watershed boundary. For flows that will occur in a currently unsewered watershed, flows were assigned to the manhole that topography indicated would be the most likely tie-in point.

Table 7 – Pump Station Flows / Capacities

Pump Station	2008 Q_{ADF} (MGD)	Q_{peak}* (MGD)
Pebble Valley	0.81	3.95
Greenmeadow	4.3	7.18
Tallgrass	UNK	UNK
Summit	0.23	3.05
Heritage Hills	0.01	0.57
Coneview	0.46	3.64
Fiddler's Creek	0.002	0.25
Badger Drive	0.11	1.08
West-Side Sub-Total	**5.92**	**19.7**
Heyer Drive	0.77	4.32
Milky Way	0.04	1.38
West Avenue	0.45	1.8
Burr Oak	0.28	2.02
Southeast Sub-Total	**1.54**	**9.52**
Grand Total	**7.46**	**29.2**

* Firm capacity unless all pumps were observed running.

Existing 25-year design flows were generated by the collection system model. Future flows were estimated as per DNR NR 110.13, which states, "*Extensions to existing sewage collection systems may be designed assuming an average design flow rate of 378 liters (100 gallons) per capita per day,*" and, "*…the peak design flow shall be determined by applying one of the following peak flow factors to the average design flow: 1. 250% of the average design flow for interceptors, main (trunk) sewers [or a peaking factor of 2.5], and sewage outfall pipes; or, 2. 400% of average design flow for sub-main and branch sewers*."

Waukesha County has completed a 2035 Land Use Plan that projects land use modifications through the year 2035, by which time the SSA is expected to be almost completely developed. This plan includes delineating the SSA into zoning classifications and projected population densities by land type. This data was sufficient to make reasonable estimates of future flows. Future flows were estimated using the following methods:

6.2.1 Septic Elimination

Septic elimination includes parcels outside the currently sewered area where a dwelling was visible. Flows were computed on a per parcel basis using the following formula:

$$Q = PF \times Household\ Size \times GPCD$$

Q = Flow (Gal/Day)
PF = Peaking Factor = 2.5
Household Size = Persons per Home = 2.5 (Waukesha County Average)
GPCD = Gallons per Capita per Day = 100 GPCD

6.2.2 Near-Future Developments

This includes regions adjacent to the currently sewered area that have been divided into parcels, but where no structures were visible. Flow estimates from these areas were computed using the same methodology as for septic elimination.

6.2.3 Far-Future Developments

Far-future development includes regions that the 2035 Land Use plan indicates are likely to be developed, but are currently open land that has not been divided into parcels. Future flows were estimated as a function of land-use projections, persons per household, housing density, and area according to the following formula:

$$Q = PF \times \text{Household Size} \times \text{Housing Density} \times \text{Area} \times GPCD$$

Q = Flow (Gal/Day)
PF = Peaking Factor = 2.5
Household Size = Persons per Home = 2.5
Housing Density = Number of Homes per Acre (Described in 2035 Land Use Plan)
Area = Land Area (acres)
GPCD = Gallons per Capita per Day = 100 GPCD

6.2.4 Future Flow Prioritization

In addition to estimating the magnitudes of future flows, consideration was given as to when these flows are likely to reach the system. Since the precise timing of development is difficult to predict, all future flows were assigned a priority of 1-4. In Figure 19, areas likely to be served in the near future (priority 1) are indicated as red, priority 2 as yellow, 3 as green, and finally 4 as blue. The locations and magnitudes of future flows into the new interceptors are also indicated in Figure 19.

In December 2007, the Southeastern Wisconsin Regional Planning Commission approved an amendment to Waukesha's SSA (Figure 20). This amendment added a 308-acre portion of Wales to Waukesha's SSA. Sanitary flow from this area would be conveyed to Waukesha's West-Side interceptor by a pump station and force main. It is estimated that the additional area would generate a daily average dry weather flow of 0.3 MGD. While this flow was not accounted for in the sizing of the West-Side force main, subsequent hydraulic analyses of the proposed sewer have determined that it has sufficient remaining capacity such that it will still provide a 25-year level of service even with the additional flow.

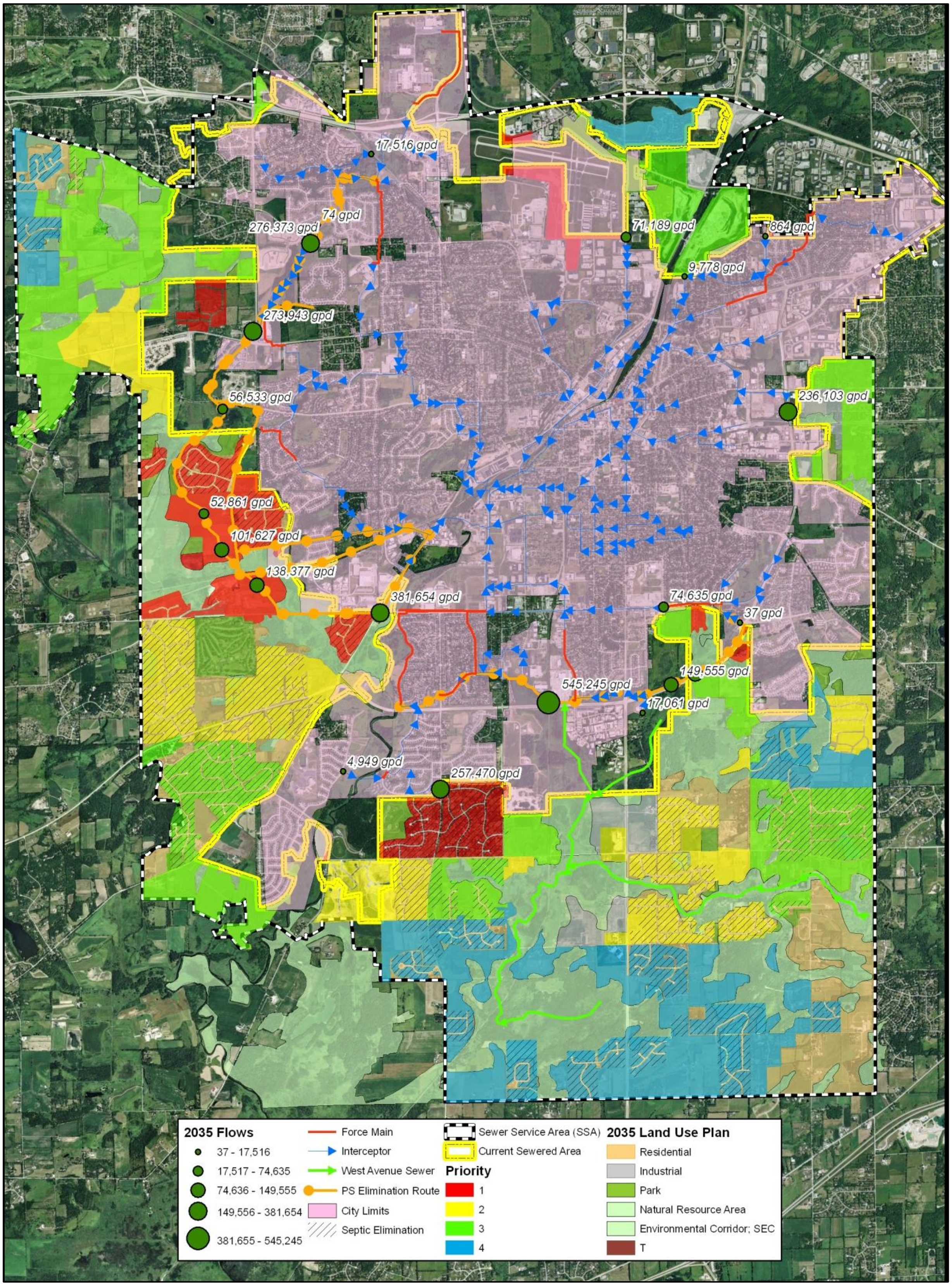


Figure 19 – Projected Future Flows

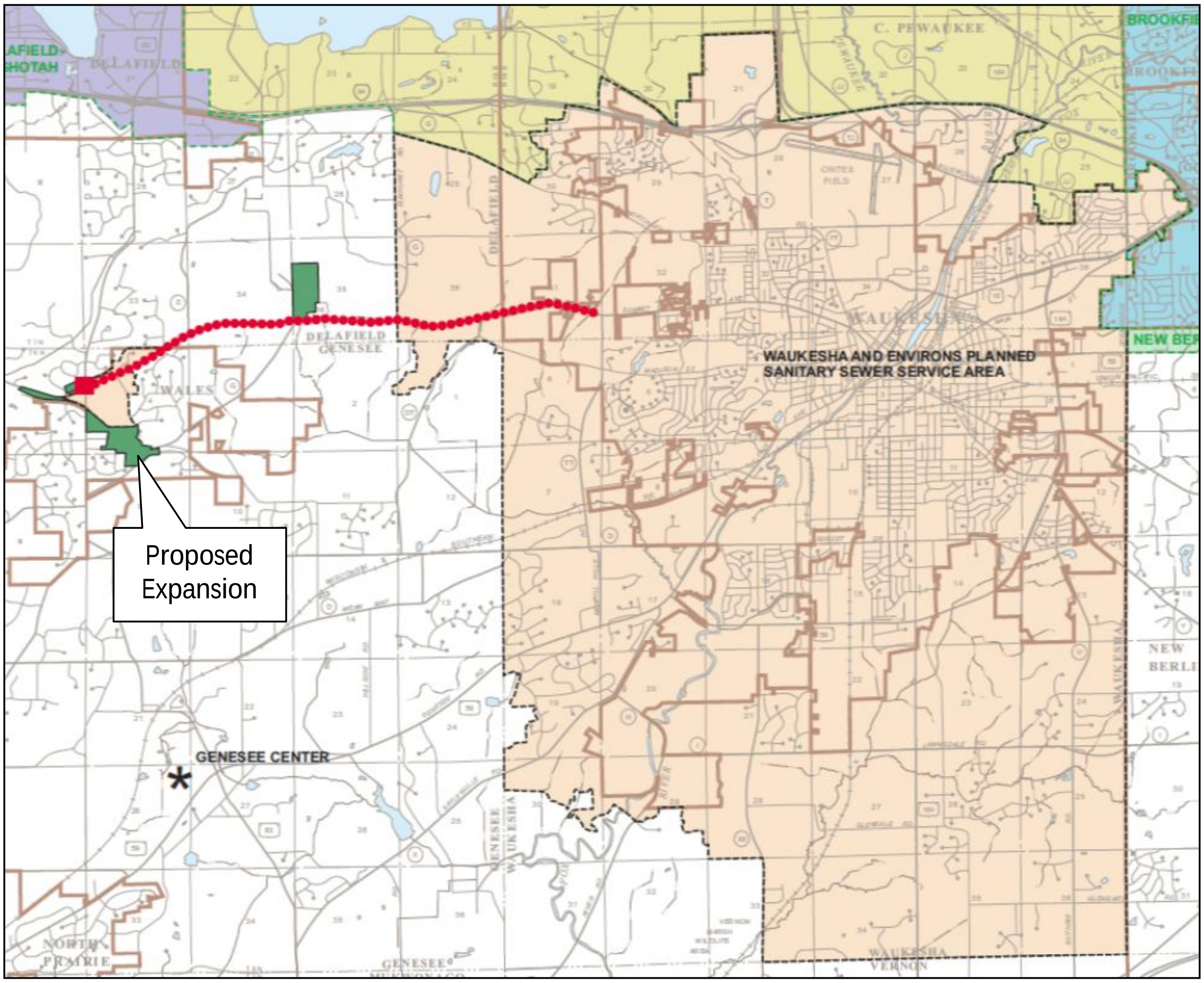


Figure 20 – 2007 SSA Amendment

6.3 SOUTHEAST INTERCEPTOR

The proposed Southeast Interceptor generally follows surface topography and existing rights-of-way (see plan and profile in Figure 21 and Figure 22). It would eliminate the following pump stations: Heyer Dr., Milky Way Rd., Burr Oak, and West Ave and terminate at the Fox Point Pump Station (PS). This station and its force main would need to be replaced to accommodate the additional flow.

The interceptor originates as a 24-inch sewer near the location of the Heyer Dr PS and increases to 30 inches before terminating at a new Fox Point PS. It was sized to maintain appropriate cover and not surcharge during the 25-year design flow. The wet weather flows from the June 18th, 2009 storm were used with the projected 2035 wastewater flows as the design flows. With a total of 4.7 inches of rain over 24 hours (2.8 inches within the first 3 hours), the storm from June 2009 was approximately a 25-year event. A 4,000-foot section of 30-inch to 36-inch sewer along the proposed route has already been constructed.

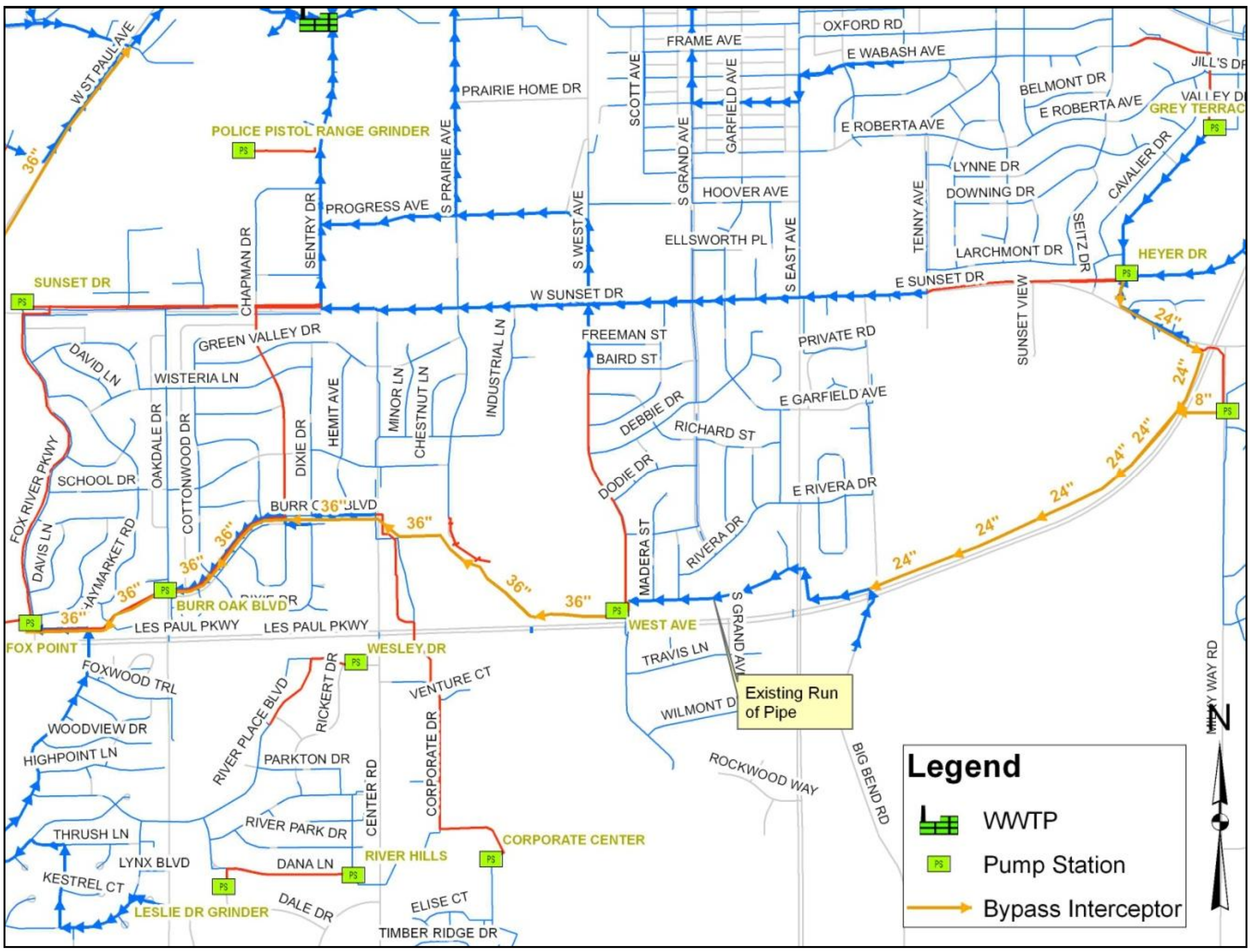


Figure 21 – Southeast Interceptor

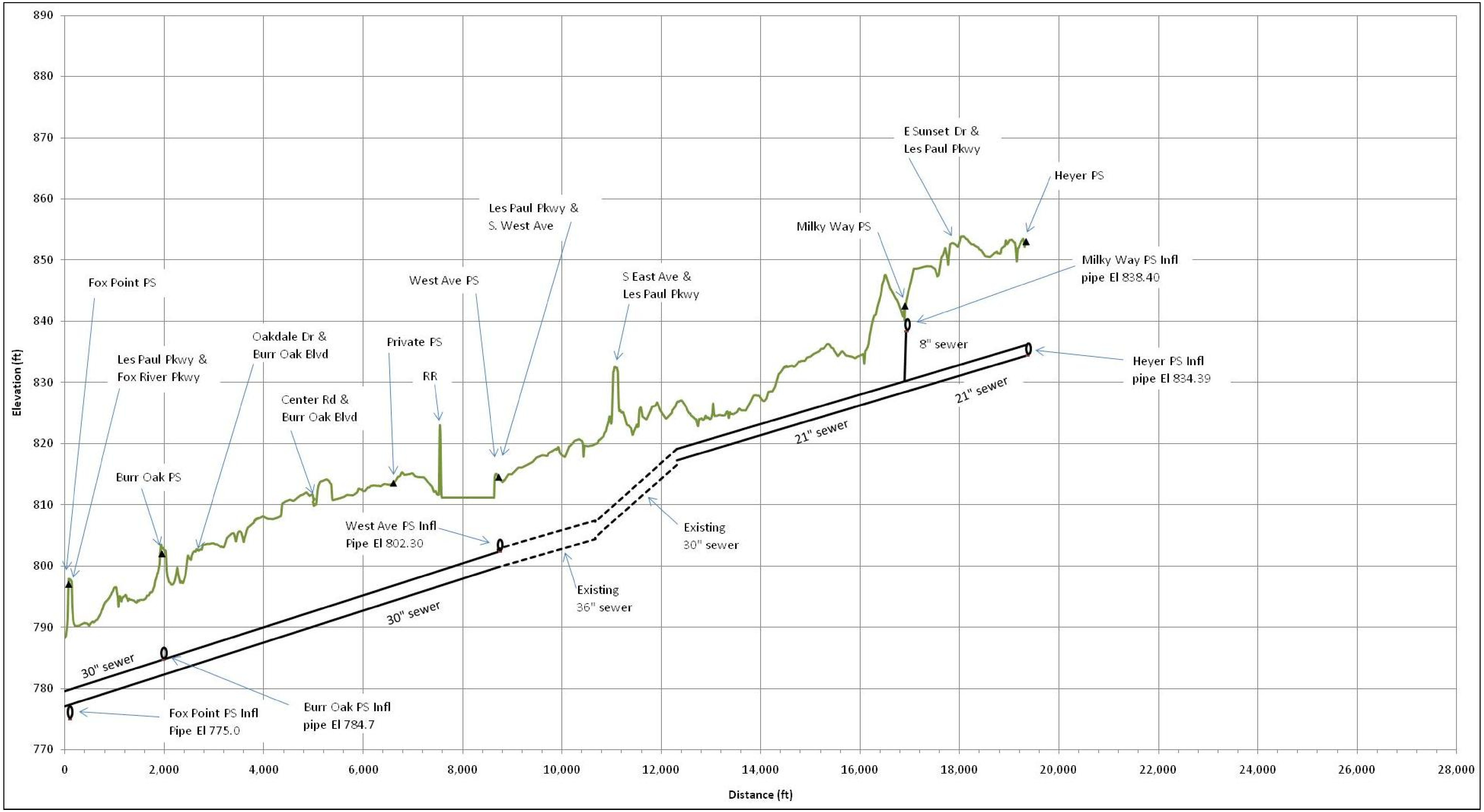


Figure 22 – Southeast Interceptor Profile

6.3.1 WEST AVENUE INTERCEPTOR

A significant unsewered area within Waukesha's SSA lies southeast of the City in the Pebble Brook watershed. Pebble Brook flows southwest towards the Fox River; conveying flows from this area to the plant will ultimately require construction of a new sewer that could tie into the proposed Southeast Interceptor at the location indicated in Figure 23. Topography likely necessitates at least two lift stations to convey these flows north. The use of lift stations (rather than pump stations) would eliminate the need for lengthy force mains.

A preliminary sizing of a West Avenue Interceptor was performed to serve the majority of the southeast service area. Design flows were calculated according to the methods described earlier. It was presumed that once the gravity sewer reached 20 feet of cover, a lift station would be required. By installing lift stations at the locations indicated in Figure 24, no force mains would be required.

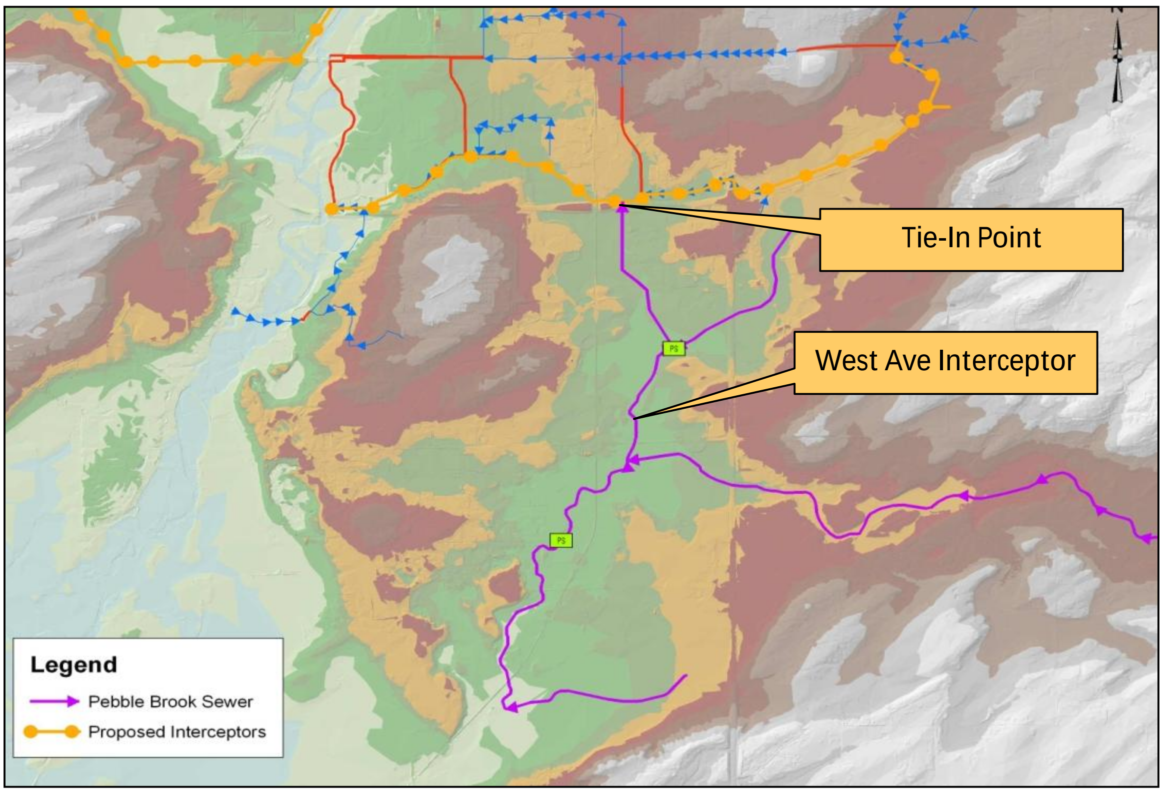


Figure 23 – Proposed Pebble Brook Interceptor

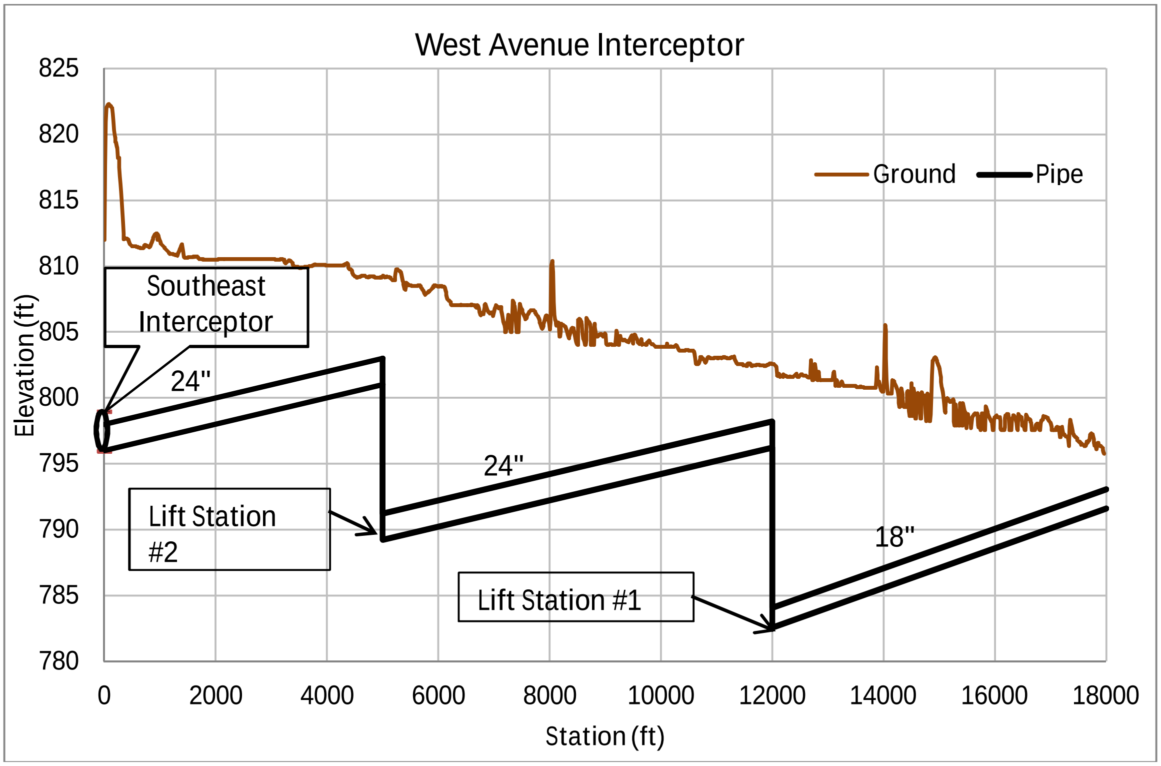


Figure 24 – Proposed West Avenue Interceptor Profile

6.4 West-Side Interceptor

During Phase I, two West-Side Interceptor routes were given consideration. The route selected for pipe sizing was slightly longer, but it is shallower and could be constructed using primarily open cut methods. A shorter route that stayed within the existing right-of-way was also considered; however, this option would require the extensive use of trenchless installation methods due to the depths involved. Wet weather flows from the June 18th, 2009 storm (a 25-year event) were used for existing flows while future flows were estimated from parcel data and the 2035 Land Use Plan.

6.4.1 Pebble Valley / Greenmeadow Pump Station Elimination Alternatives

Due to the length of sewer involved, the elimination of the Pebble Valley and Greenmeadow Pump Stations was found to be incrementally more expensive than the other stations along this route. Therefore, the West-Side Interceptor was sized for four different alternatives for eliminating Pebble Valley and/or Greenmeadow. The precise sewer route had little effect on sewer sizing, although it significantly affected costs. Depending on the selected route, Badger Drive PS or the majority of the MacArthur Road PS force main could be eliminated.

Alternative #1 could eliminate up to seven pump stations, including Pebble Valley. In Phase I, the bypass was sized to match the capacity of the Pebble Valley PS; however, the Pebble Valley PS may be undersized, and the gravity flows were found to overload the upstream section of the proposed interceptor. Therefore, the size of this segment of sewer was increased from an 18-inch to a 24-inch to accommodate the simulated 25-year wet weather flows. A sensitivity analysis was performed to ascertain the extent to which reducing wet weather flows might reduce the size and cost of the proposed interceptor.

Alternative #2 eliminates up to eight pump stations, including both Pebble Valley and Greenmeadow. An 18-inch sewer currently diverts excess flow from the Greenmeadow PS to the Coneview PS. This sewer would need to be upsized to accommodate all Greenmeadow flows even if Pebble Valley is diverted into the proposed interceptor.

Alternative #3 eliminates the Greenmeadow pump station but not the Pebble Valley station. This option also requires upsizing the sewer that currently diverts flow from Greenmeadow to Coneview.

Alternative #4 originates at the Summit Pump Station and does not include the elimination of the Pebble Valley or Greenmeadow stations. With this option, up to five other pump stations are eliminated.

The preceding four alternatives are summarized in Table 8.

Table 8 – West-Side Interceptor Pump Station Elimination Alternatives

Station/Alternative #	1	2	3	4
Pebble Valley	X	X		
Green Meadow		X	X	
Tall Grass	X	X		
Summit Ave.	X	X	X	X
Coneview	X	X	X	X
Badger Dr.*	X	X	X	X
Madison St.	X	X	X	X
Fiddler's Creek	X	X	X	X
Total	7	8	6	5

*Feasible only if Route #1 is selected.

The sewer routes that have been given consideration are shown in Figure 25. While Route 1 is the longest, it is also the shallowest and has the greatest potential to collect future flows. This route also has the potential to "piggyback" the State Bypass project. Route 2 follows the existing ROW south along Merrill Hills road and east along Mac Arthur Road. This route would require the extensive use of trenchless technologies due to the depths of cover involved. Route 3 parallels the Glacial Drumlin State Trail; this route has not been given serious consideration and is not included in this analysis.

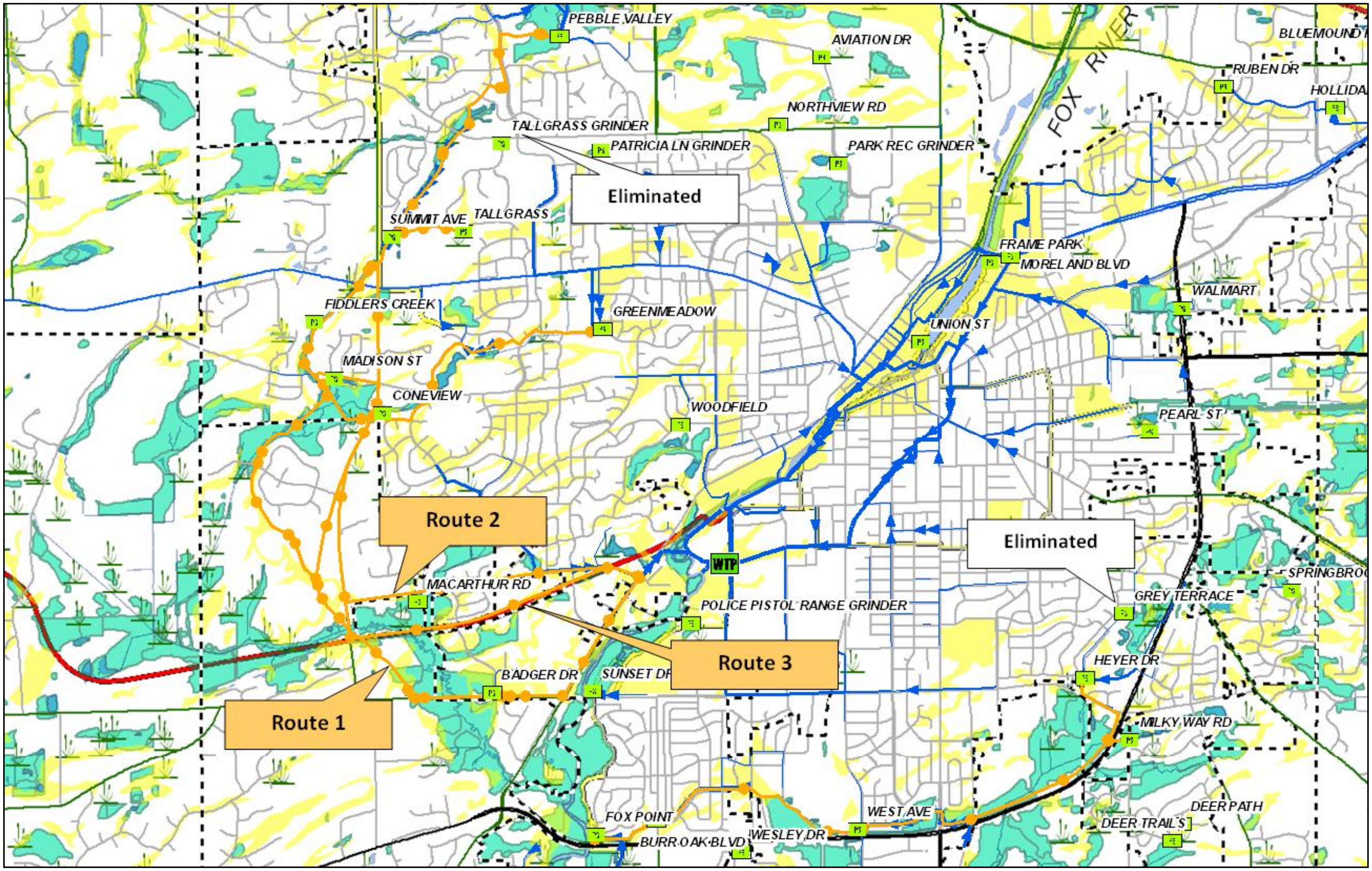


Figure 25 – Interceptor Routes

In order to simplify comparing the four elimination alternatives, the routes were divided into four sections as indicated in Figure 26 and Figure 27. Each section consists of a pipe of uniform size and slope. Profiles of these routing alternatives and the Greenmeadow sewer have been included as Figures 9 through 13.

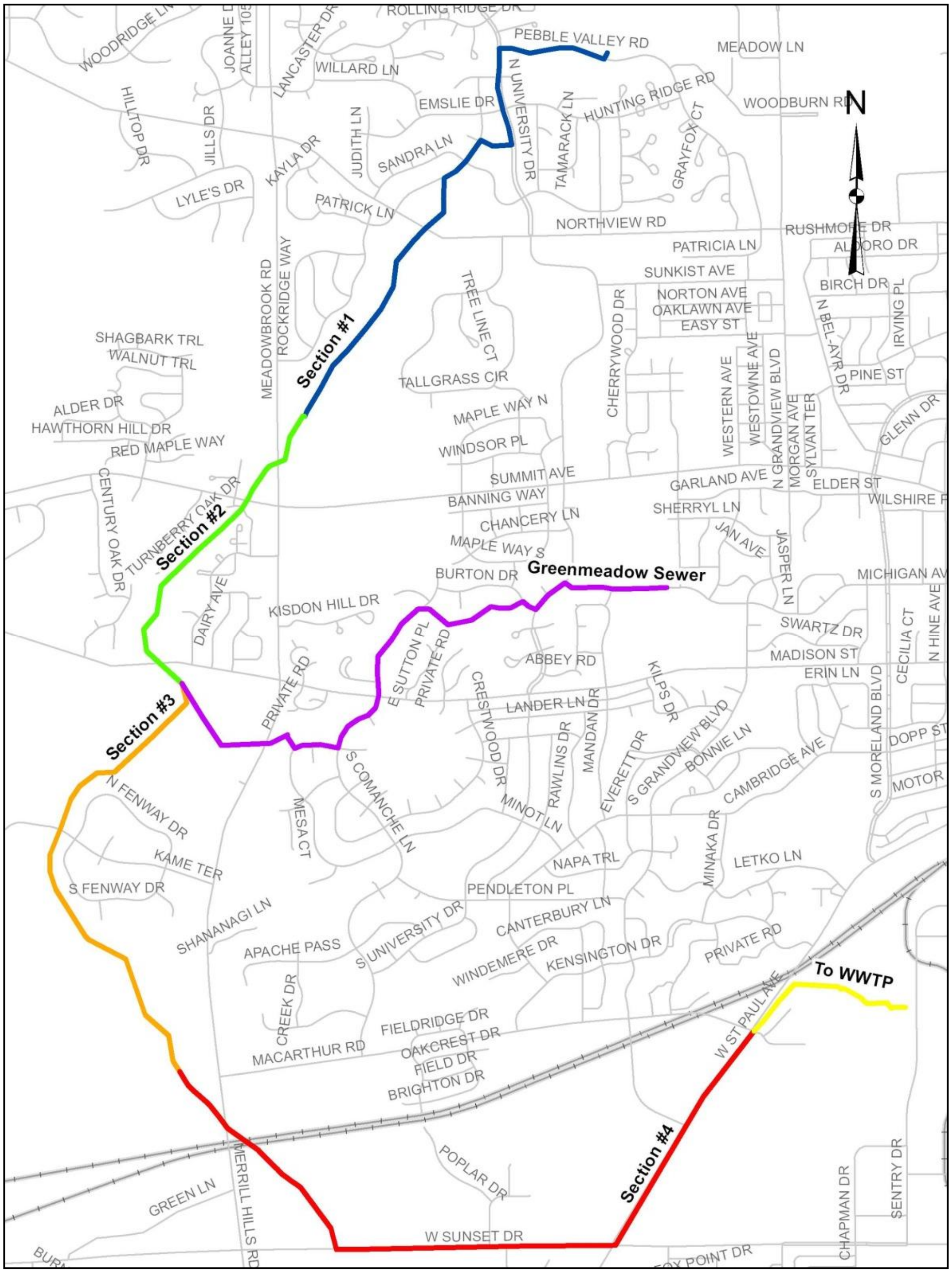


Figure 26 – Route #1 West-Side Interceptor Sections

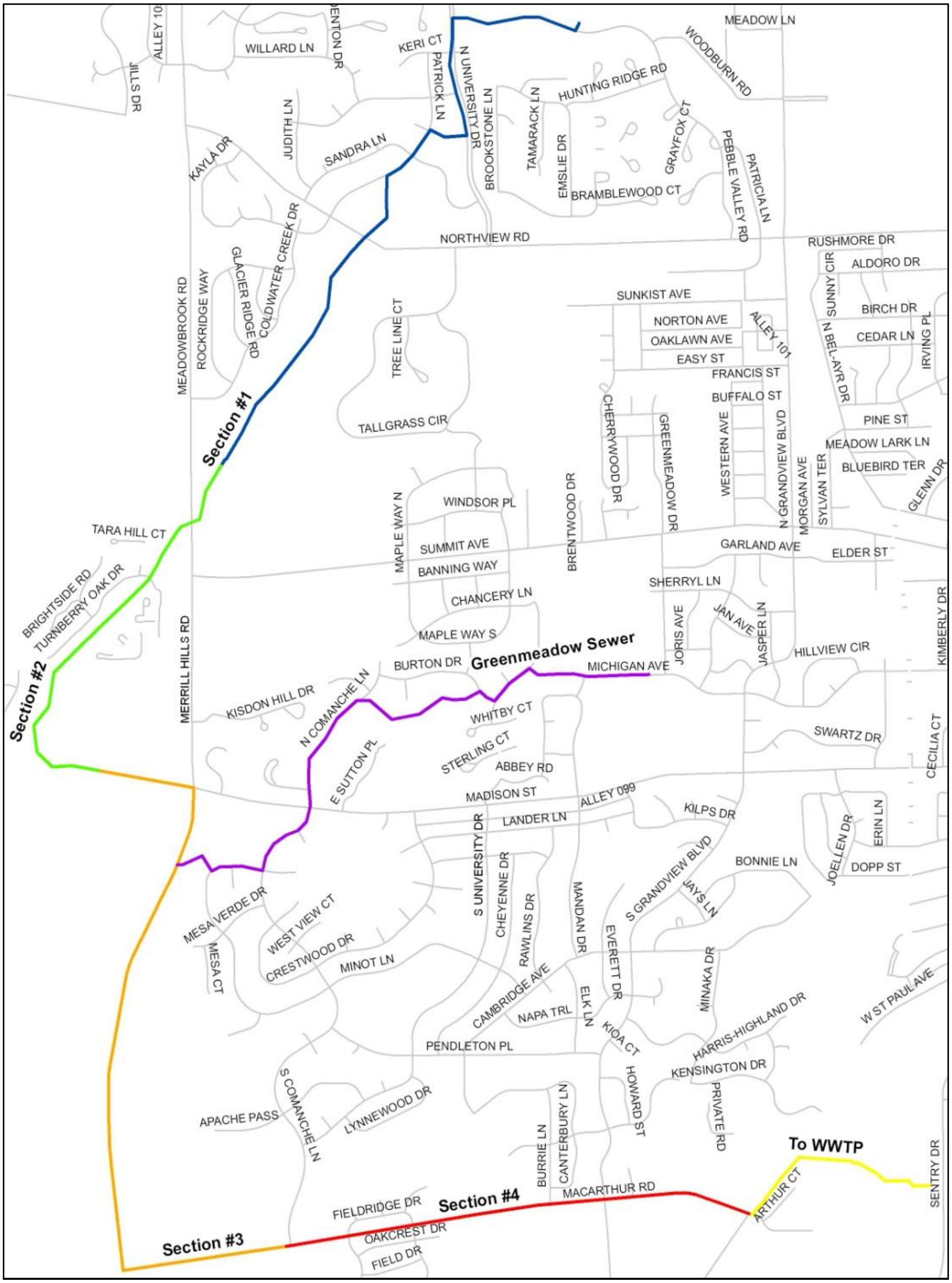


Figure 27 – Route #2 West-Side Interceptor Sections

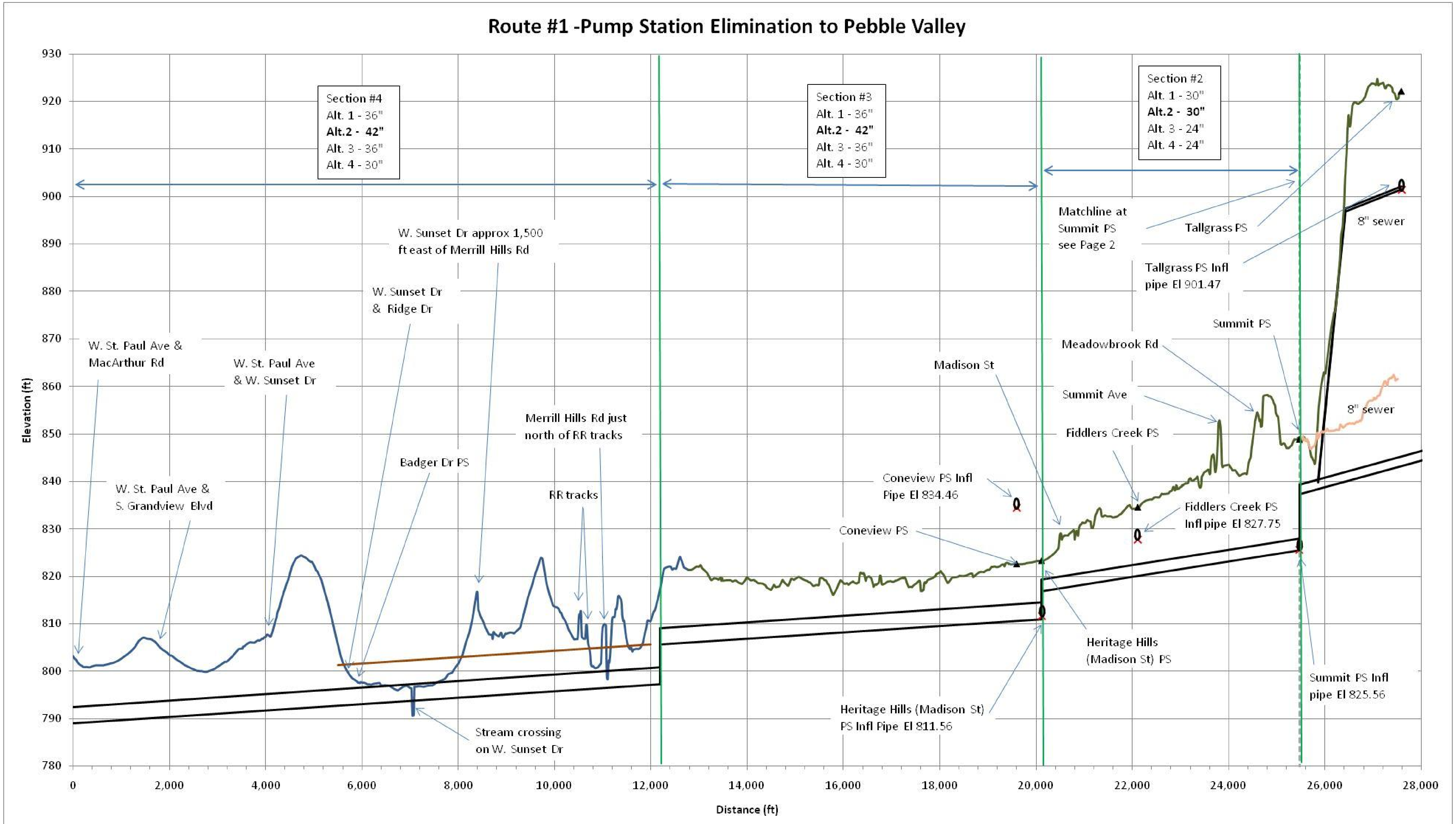


Figure 28 – Route #1 Interceptor Profile, Sections 2-4

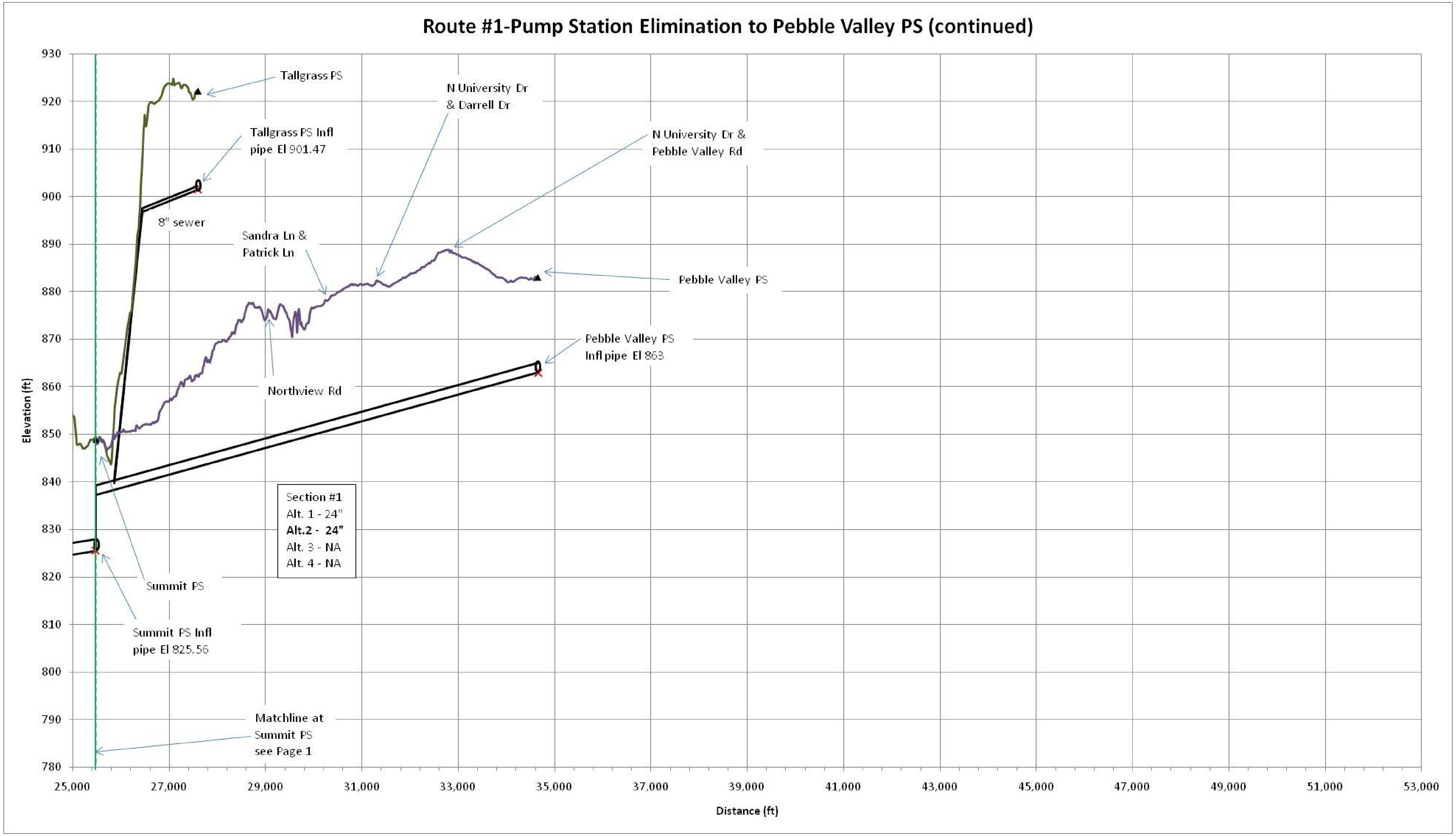


Figure 29 – Route #1 Interceptor Profile, Section 1

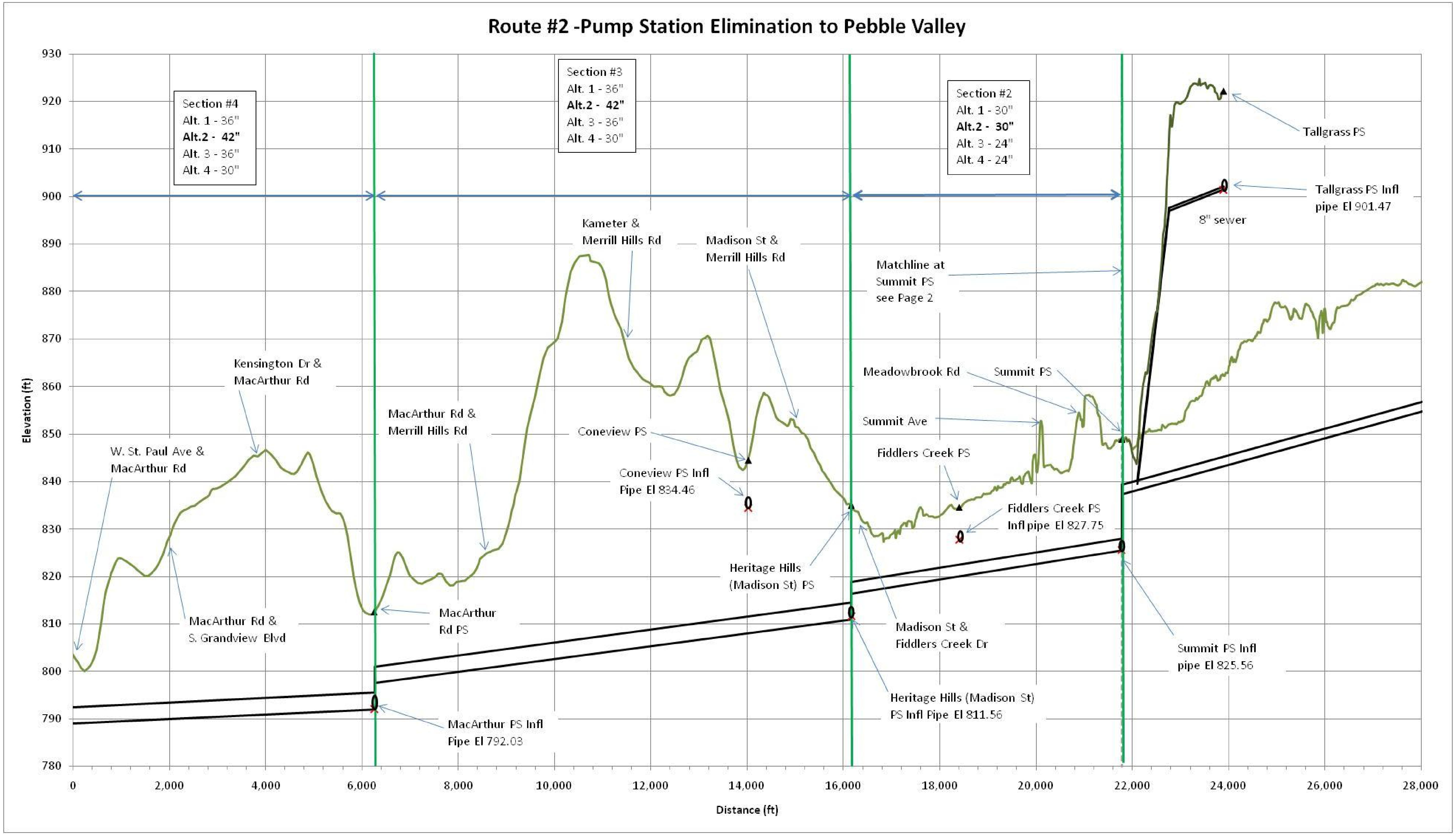


Figure 30 – Route #2 Interceptor Profile, Sections 2-4

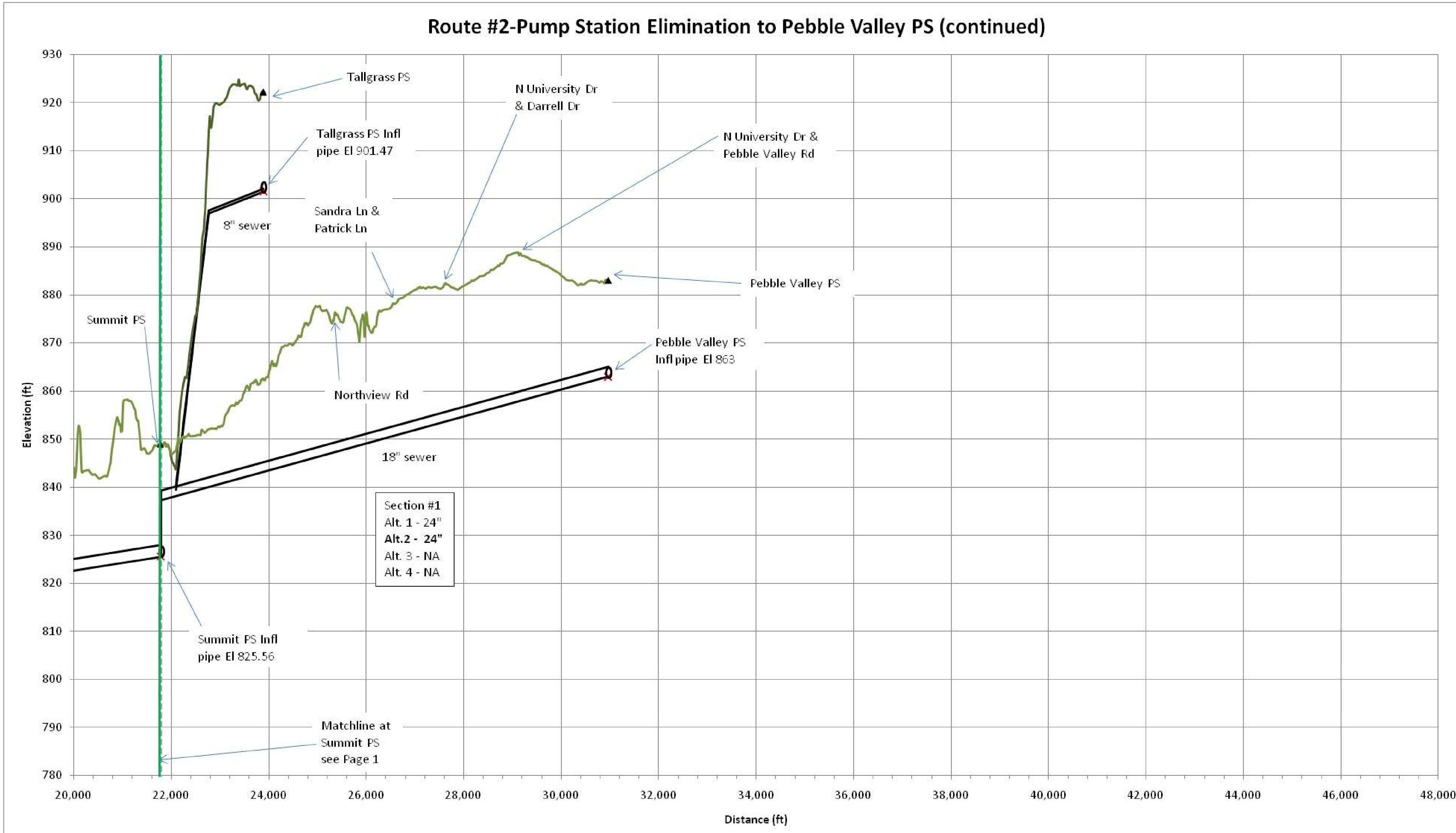


Figure 31 – Route #2 Interceptor Profile, Section 1

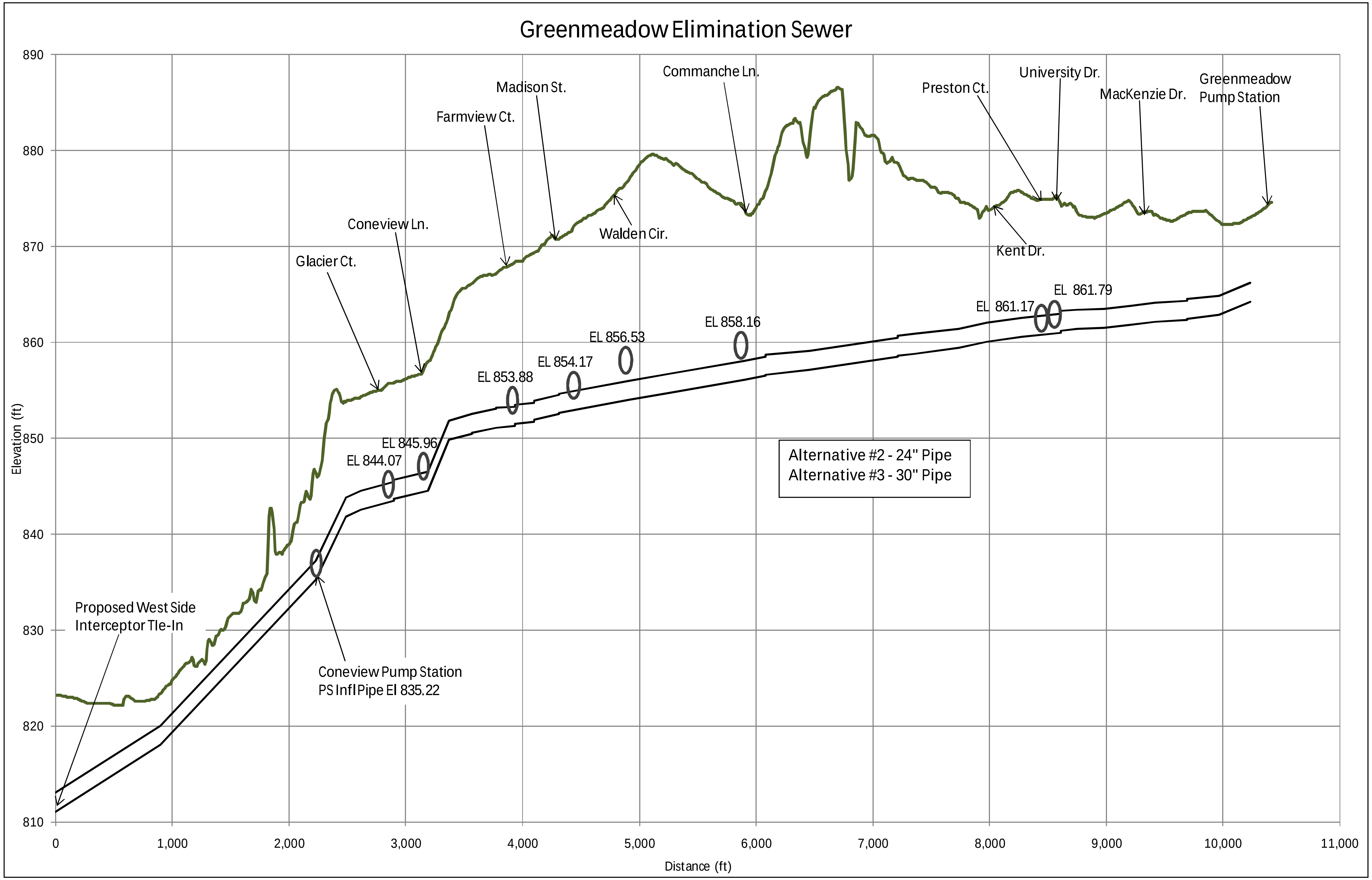


Figure 32 – Greenmeadow Sewer

6.5 Future Flows and I/I Reduction Evaluation

With the addition of the future flow estimates from Section 6.2, the 2035 flows reaching the treatment plant are estimated to be:

- Q_{DWF} = 12.8 MGD
- Q_{WWF} = 55 MGD

Collection system improvements should be designed for the ultimate sewer service area flows (as currently understood). However, as the WWTP improvements may be designed for a 20-year expansion period, according to NR 110 regulations.

Future collection system flows will be developed over a period of time as development and annexation increases the sewered population. AECOM continues to refine the model and flow projections, therefore we recommend that the once the model is finalized, a series of final plant influent hydrographs be developed for incoming plant flows for varying recurrence interval storms. With these hydrographs, the most cost effective flow management alternative can be selected. This may include; 1) Increasing the plant peak flow capacity, 2) Reduction of the peak flow via reduction of infiltration and inflow in the collection system, 3) Remote storage within the collection system, and/or 4) Storage at the plant by increasing the influent pump sizing and utilizing the re-furbished trickling filter as excess flow storage basins (4.6 MG, total). The determination of the optimum means to convey/treat the additional peak hourly flow has yet to be determined and will be addressed with the next WWTP Facilities Plan.

Inflow & infiltration (I/I) reduction was evaluated as a potential capital cost saving measure. I/I reduction methods may include manhole rehabilitation/replacement, sewer lining/replacement, lateral rehab/replacement, etc. The cost of I/I removal typically increases sharply with higher removal rates. The improvement costs required to achieve even a moderate level I/I reduction often exceed savings in conveyance and treatment. Furthermore, reliable cost estimates for I/I reduction are difficult to obtain.

To evaluate the effects of I/I reduction, a new model scenario was created in which I/I was reduced by 30%. This is generally the upper limit of what can practically be eliminated. West-Side Alternative #2, Route 1 was used to estimate the decrease in construction costs resulting from I/I reduction because it eliminates the most pump stations and would therefore likely yield the greatest corresponding reduction in capital construction costs.

This simulation reduced most of the proposed sewer sections by one pipe diameter, reducing construction costs by approximately $2.5 million. Achieving a 30% reduction in I/I in those areas served by the proposed interceptor would likely cost far more than this. Therefore, I/I reduction does not appear to be a significant capital cost savings measure. However, this cost-benefit analysis considers only capital cost savings; it does not consider reductions in treatment costs.

6.6 Cost Calculations

6.6.1 Construction Costs

Table 9 summarizes the capital construction costs of the pump station elimination alternatives and routes. More cost breakdowns are included as Table 10 through Table 14.

Table 9 – Interceptor Construction Cost Matrix

Pebble Creek (West-Side) Interceptor				
	Without Future Flows		With Future Flows	
Alternative #	Route #1	Route #2	Route #1	Route #2
1	$12,185,369	$23,444,811	$13,008,431	$24,474,313
2	$18,234,769	$28,946,370	$19,057,832	$29,975,872
3	$12,853,123	$26,144,513	$13,676,185	$27,174,015
4	$7,057,332	$20,098,507	$7,057,332	$20,098,507

Southeast Interceptor		
Component	Without Future Flows	With Future Flows
Sewer	$5,482,600	$6,846,200
Fox Point PS	$3,000,000	$3,000,000
Force Main	$1,250,000	$1,250,000
Total	$9,732,600	$11,096,200

Table 10 – West-Side Interceptor Open Cut Unit Costs (Route 1)

Section	Average Depth (ft)	Pipe Size (in)				Unit Costs ($/L.F.)			
		Alt #1	Alt #2	Alt #3	Alt #4	Alt #1	Alt #2	Alt #3	Alt #4
1	14.22	24	24	NA	NA	$192.97	$192.97	NA	NA
2	14.24	30	30	24	24	$347.18	$347.18	$295.17	$295.17
3	8.62	36	42	36	30	$240.82	$282.39	$220.92	$203.67
4	11.00	36	42	36	30	$311.71	$389.72	$332.46	$265.12

Table 11 – West-Side Interceptor Open Cut Unit Costs (Route 2)

Section	Average Pipe Depth (ft)	Pipe Size (in)				Unit Cost ($/L.F.)			
		Alt #1	Alt #2	Alt #3	Alt #4	Alt #1	Alt #2	Alt #3	Alt #4
1	21.56	24	24	-	-	$228.36	$228.36	-	-
2	15.47	30	30	24	24	$234.25	$234.25	$199.00	$199.00
3	39.08	36	42	36	30	$454.11	$526.17	$454.11	$388.94
4	36.36	36	42	36	30	$477.79	$551.54	$477.79	$410.93

Note: Tunneling was presumed for depths greater than 30 feet. A tunneling unit cost of $1500/LF was utilized (prices quoted ranged from $1200-$1800/LF).

Table 12 – West-Side Interceptor Route 1 Cost Summary (with Future Flows)

Section	Open Cut Footage (LF)	Tunneling Footage (LF)	Alt 1	Alt 2	Alt 3	Alt 4
1	9,190	0	$1,773,407	$1,775,710	NA	NA
2	5,461	0	$1,859,632	$1,978,865	$433,310	$433,310
3	7,869	0	$1,897,828	$2,228,188	$1,658,360	$1,605,275
4	12,188	0	$4,217,428	$5,420,373	$4,625,799	$3,607,281
Interceptor Totals	34,708	0	**$9,748,295**	**$11,403,136**	**$6,717,469**	**$5,645,865**
		Additional Sewer Costs $	$823,062	$4,803,912	$5,279,349	$0
		SubTotal	**$10,571,357**	**$16,207,048**	**$11,996,818**	**$5,645,865**
		5% Engineering	$487,415	$570,157	$335,873	$282,293
		20% Contingency	$1,949,659	$2,280,627	$1,343,494	$1,129,173
		Total Estimated Cost	**$13,008,431**	**$19,057,832**	**$13,676,185**	**$7,057,332**

Table 13 – West-Side Interceptor Route 1 Cost Summary (without Future Flows)

Section	Open Cut Footage (LF)	Tunneling Footage (LF)	Alt 1	Alt 2	Alt 3	Alt 4
1	9,190	0	$1,773,407	$1,775,710	NA	NA
2	5,461	0	$1,859,632	$1,978,865	$433,310	$433,310
3	7,869	0	$1,897,828	$2,228,188	$1,658,360	$1,605,275
4	12,188	0	$4,217,428	$5,420,373	$4,625,799	$3,607,281
Interceptor Totals	34,708	0	**$9,748,295**	**$11,403,136**	**$6,717,469**	**$5,645,865**
		Additional Sewer Costs $	$0	$3,980,850	$4,456,287	$0
		SubTotal	**$9,748,295**	**$15,383,985**	**$11,173,756**	**$5,645,865**
		5% Engineering	$487,415	$570,157	$335,873	$282,293
		20% Contingency	$1,949,659	$2,280,627	$1,343,494	$1,129,173
		Total Estimated Cost	**$12,185,369**	**$18,234,769**	**$12,853,123**	**$7,057,332**

Note: “Additional Sewer Costs” include the Greenmeadow sewer (Alternatives 2 and 3), and upsizing the river crossing near the WTP (see Figure 26and Figure 27).

Table 14 – West-Side Interceptor Route 2 Cost Summary (with Future Flows)

Section	Total Open Cut Footage (LF)	Total Tunneling Footage (LF)	Alternatve 1 Cost $	Alternatve 2 Cost $	Alternatve 3 Cost $	Alternatve 4 Cost $
1	9,194	0	$2,105,909	$2,105,909	$0	$0
2	5,408	209	$1,521,929	$1,521,929	$1,331,313	$1,331,313
3	3,828	6068	$9,025,436	$9,301,283	$9,025,436	$8,775,963
4	1,960	4300	$6,102,575	$6,247,125	$6,102,575	$5,971,530
Interceptor Totals	20,389	10577	**$18,755,849**	**$19,176,246**	**$16,459,323**	**$16,078,805**
		Additional Sewer Costs $	$823,602	$4,804,452	$5,279,889	$0
		SubTotal	**$19,579,451**	**$23,980,698**	**$21,739,212**	**$16,078,805**
		5% Engineering	$978,973	$1,199,035	$1,086,961	$803,940
		20% Contingency	$3,915,890	$4,796,140	$4,347,842	$3,215,761
		Total Estimated Cost	**$24,474,313**	**$29,975,872**	**$27,174,015**	**$20,098,507**

6.6.2 ENERGY SAVINGS

Table 15 below summarizes the annual and 20-year energy savings in pumping costs these interceptors provide. Please note that the energy savings the southeast interceptor provides will be substantially offset by the energy costs of the new Fox Point pump station.

Energy savings alone do not appear to justify the capital costs of constructing the new interceptors. However, this analysis does not include the labor and equipment costs that these pump stations incur.

Table 15 – Energy Savings

			20-Year Totals	
Station	**Annual Usage (kWh)**	**Annual Electric Charges**	**Usage (MWh)**	**Electric Charges**
Badger Dr	19,161	$ 2,500	383	$ 49,995
Coneview	56,000	$ 7,031	1,120	$ 140,629
Fiddler's Creek	1,277	$ 356	26	$ 7,123
Heritage Hills (Madison)	6,453	$ 973	129	$ 19,468
Greenmeadow	183,667	$ 20,798	3,673	$ 415,967
Pebble Valley	255,208	$ 29,018	5,104	$ 580,358
Summit	48,419	$ 5,914	968	$ 118,286
Tallgrass	15,542	$ 2,066	311	$ 41,319
Pebble Creek Total	**585,727**	**$ 68,657**	**11,715**	**$1,373,145**
Burr Oak	31,907	$ 3,997	638	$ 79,942
Heyer Dr	329,452	$ 34,805	6,589	$ 696,091
Milky Way	4,472	$ 739	89	$ 14,772
West Ave	83,652	$ 10,051	1,673	$ 201,016
* **Southeast Total**	**449,483**	**$ 49,591**	**8,990**	**$ 991,821**

CHAPTER VII – 20-YEAR CAPITAL IMPROVEMENT PLAN

Three Capital Improvement Plans (CIPs) have been prepared for each of 3 pump station elimination alternatives. These are:

- Table 16 – 20-Yr Capital Improvement Plan (PS Elimination, Route 1) – This alternative presumes that Route 1 (Figure 26) and Alternative 2 (Table 9) will be selected and the Southeast Interceptor will be constructed.
- Table 17 – 20-Yr Capital Improvement Plan (PS Elimination, Route 2) – This alternative presumes that Alternative 2 (Table 9) will be selected, but that the West-Side interceptor will follow Route 2 (Figure 27), a more expensive option.
- Table 18 – 20-Yr Capital Improvement Plan (No PS Elimination) – This alternative presumes that neither of the proposed pump station elimination interceptors will be constructed. Under this scenario, the West Avenue pump station and force main will have to be replaced to accommodate the additional flow from the West Avenue interceptor (Section 6.3.1) that will serve the southeast region. It should be noted, that this option does <u>not</u> include the costs to maintain, and potentially upgrade, those stations that would otherwise be eliminated by the West-Side Interceptor.

For all three CIP versions, annual sewer maintenance and rehabilitation expenses are consistent with Waukesha's CMOM Plan. Pump station repairs and upgrades are based on input received from City personnel. It is presumed that the remaining ferrous force mains will ultimately be replaced with PVC mains as funding allows. Force main replacement has been postponed until at least 2018 as the Force Main Risk Assessment and subsequent testing have determined that the ferrous mains should have sufficient remaining useful life.

Table 16 – 20-Yr Capital Improvement Plan (PS Elimination, Route 1)

	2011	2012	2013	2014	2015	2016	2017	2018	2019	2020	2021	2022	2023	2024	2025	2026	2027	2028	2029	2030	Total
Collection System																					
Sewer Rehabilitation																					
Sanitary Sewer Rehabilitation - Minor Sewers	$ 1,500,000	$ 1,500,000	$ 1,500,000	$ 1,500,000	$ 1,500,000	$ 1,500,000	$ 1,500,000	$ 1,500,000	$ 1,500,000	$ 1,500,000	$ 1,500,000	$ 1,500,000	$ 1,500,000	$ 1,500,000	$ 1,500,000	$ 1,500,000	$ 1,500,000	$ 1,500,000	$ 1,500,000	$ 1,500,000	**$ 30,000,000**
Manhole Rehabilitation	$ 300,000	$ 324,000	$ 324,000	$ 324,000	$ 324,000	$ 324,000	$ 324,000	$ 324,000	$ 324,000	$ 324,000	$ 324,000	$ 324,000	$ 54,000	$ 54,000	$ 54,000	$ 54,000	$ 54,000	$ 54,000	$ 54,000	$ 54,000	**$ 4,296,000**
Total Sewer Rehabilitation	$ 1,800,000	$ 1,824,000	$ 1,824,000	$ 1,824,000	$ 1,824,000	$ 1,824,000	$ 1,824,000	$ 1,824,000	$ 1,824,000	$ 1,824,000	$ 1,824,000	$ 1,824,000	$ 1,554,000	$ 1,554,000	$ 1,554,000	$ 1,554,000	$ 1,554,000	$ 1,554,000	$ 1,554,000	$ 1,554,000	**$ 34,296,000**
Southeast Interceptor																					
Design (5% of construction)	$ 519,582																				**$ 519,582**
Replace Fox Point Pump Station		$ 3,000,000																			**$ 3,000,000**
Replace Fox Point Force Main			$ 1,437,500																		**$ 1,437,500**
30" interceptor from Fox Point to Burr Oak			$ 588,800																		**$ 588,800**
30" interceptor from Burr Oak to West Ave				$ 2,137,850																	**$ 2,137,850**
21" sewer to Les Paul Pkwy & Legend Hill Ln					$ 1,624,388																**$ 1,624,388**
8" sewer from Interceptor to Milky Way PS					$ 65,550																**$ 65,550**
21" Interceptor to Heyer Dr. PS						$ 1,537,550															**$ 1,537,550**
Southeast Interceptor Total	$ 519,582	$ 3,000,000	$ 2,026,300	$ 2,137,850	$ 1,689,938	$ 1,537,550															**$ 10,911,220**
West-Side Interceptor (Route 1)																					
Design (5% of construction)	$ 914,731																				**$ 914,731**
42" interceptor from intersection of Les Paul Pkwy. & Grandview Blvd. to Badger Dr. PS							$ 3,780,781														**$ 3,780,781**
Build 42" interceptor from Badger Dr. PS to Coneview PS Tie-In								$ 4,755,012													**$ 4,755,012**
Build 30" Interceptor to to Madison St. PS								$ 161,820													**$ 161,820**
Build 30" interceptor to Fiddler's Creek PS								$ 627,979													**$ 627,979**
Build 30" sewer to Summit Ave PS									$ 1,320,492												**$ 1,320,492**
Build 30" sewer toTallgrass PS Tie In									$ 21,508												**$ 21,508**
Build 8" sewer to Tall Grass PS									$ 286,812												**$ 286,812**
Upsize Sewer from Interceptor Tie in to WWTP										$ 918,468											**$ 918,468**
Build 24" sewer to Pebble Valley											$ 1,979,122										**$ 1,979,122**
Build 24" sewer from interceptor through Coneview PS to Greenmeadow PS												$ 4,442,628									**$ 4,442,628**
West-Side Interceptor Total	$ 914,731						$ 3,780,781	$ 5,544,811	$ 1,628,812	$ 918,468	$ 1,979,122	$ 4,442,628									**$ 19,209,353**
West Avenue Interceptor																					
Design (5% of construction for pipes, 15% for PS)				$ 140,520	$ 228,420			$ 121,380	$ 170,940												**$ 661,260**
24-inch Sewer					$ 2,810,400																**$ 2,810,400**
Lift Station #2						$ 1,522,800															**$ 1,522,800**
18-inch Mill Creek Sewer									$ 2,427,600												**$ 2,427,600**
18-inch Pebble Brook Sewer										$ 927,600											**$ 927,600**
Lift Station #1										$ 830,400											**$ 830,400**
West Avenue Interceptor Total				$ 140,520	$ 3,038,820	$ 1,522,800	$ -	$ 121,380	$ 2,598,540	$ 1,758,000											**$ 9,180,060**
Pump Stations & Force Mains																					
Wesley Drive PS & FM Elimination	$ 130,000																				**$ 130,000**
Pump Station Flood Protection (Sunset Dr, Coneview, Summit Ave, and West Ave)	$ 200,000																				**$ 200,000**
Bluemound (Control panel, valve vault)	$ 200,000																				**$ 200,000**
Badger Drive plumbing upgrade, pumps, piping, and valves moved out of wetwell)			$ 100,000																		**$ 100,000**
General Electric (Under way)	$ 330,000																				**$ 330,000**
Hollidale (Valve vault & control panel)		$ 200,000																			**$ 200,000**
Greenmeadow (Replace 2 pumps, re-wind motors, internal plumbing, force main piping, asbestos removal)			$ 50,000			$ 150,000															**$ 200,000**
Pearl (Electrical upgrade)					$ 50,000																**$ 50,000**
MacArthur (Control panel, valve vault)		$ 300,000																			**$ 300,000**
Fox Point (Control panel)		$ 50,000																			**$ 50,000**
Golf Road (Increase capacity / replace)				$ 350,000																	**$ 350,000**
Northview (Hatch, repair drive, force main)	$ 75,000																				**$ 75,000**
Woodfield (New wet well, valve vault)				$ 250,000																	**$ 250,000**
Walmart (Upgrade control panel)			$ 50,000																		**$ 50,000**
Patricia (Generator)					$ 50,000																**$ 50,000**
Pebble Valley (Generator)				$ 200,000																	**$ 200,000**
Other Pump Station Upgrades / Rehab ($40/yr/gpm)							$ 250,000	$ 250,000	$ 250,000	$ 250,000	$ 250,000	$ 250,000	$ 250,000	$ 250,000	$ 250,000	$ 250,000	$ 250,000	$ 250,000	$ 250,000	$ 250,000	**$ 3,500,000**
Replace Ferrous Force Mains																					
Ruben Drive												$ 1,135,000									**$ 1,135,000**
Northview													$ 130,000								**$ 130,000**
Wal-Mart													$ 220,000								**$ 220,000**
Sunset Drive														$ 700,000							**$ 700,000**
MacArthur Road															$ 415,000						**$ 415,000**
Springbrook																$ 740,000					**$ 740,000**
Hollidale																		$ 13,000			**$ 13,000**
Woodfield																		$ 130,000			**$ 130,000**
Bluemound																		$ 94,000			**$ 94,000**
Total Pump Stations & Force Mains	$ 935,000	$ 550,000	$ 200,000	$ 800,000	$ 100,000	$ 150,000	$ 250,000	$ 250,000	$ 250,000	$ 250,000	$ 250,000	$ 1,385,000	$ 600,000	$ 950,000	$ 665,000	$ 990,000	$ 250,000	$ 487,000	$ 250,000	$ 250,000	**$ 9,812,000**
CMOM																					
Manhole Inspections	$ 56,000	$ 56,000	$ 56,000	$ 56,000	$ 24,000	$ 24,000	$ 24,000	$ 24,000													**$ 320,000**
CCTV	$ 68,640	$ 76,427	$ 52,765	$ 51,237	$ 51,143	$ 51,110	$ 53,491	$ 61,612													**$ 466,425**
Flusher Truck with Camera		$ 200,000																			**$ 200,000**
Camera Truck				$ 200,000																	**$ 200,000**
CMOM Total	$ 124,640	$ 332,427	$ 108,765	$ 307,237	$ 75,143	$ 75,110	$ 77,491	$ 85,612	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	**$ 1,186,425**
Collection System Total	**$ 4,293,953**	**$ 5,706,427**	**$ 4,159,065**	**$ 5,209,607**	**$ 6,727,901**	**$ 5,109,460**	**$ 5,932,272**	**$ 7,825,802**	**$ 6,301,352**	**$ 4,750,468**	**$ 4,053,122**	**$ 7,651,628**	**$ 2,154,000**	**$ 2,504,000**	**$ 2,219,000**	**$ 2,544,000**	**$ 1,804,000**	**$ 2,041,000**	**$ 1,804,000**	**$ 1,804,000**	**$ 84,595,058**

Table 17 – 20-Yr Capital Improvement Plan (PS Elimination, Route 2)

	2011	2012	2013	2014	2015	2016	2017	2018	2019	2020	2021	2022	2023	2024	2025	2026	2027	2028	2029	2030	Total
Collection System																					
Sewer Rehabilitation																					
Sanitary Sewer Rehabilitation - Minor Sewers	$ 1,500,000	$ 1,500,000	$ 1,500,000	$ 1,500,000	$ 1,500,000	$ 1,500,000	$ 1,500,000	$ 1,500,000	$ 1,500,000	$ 1,500,000	$ 1,500,000	$ 1,500,000	$ 1,500,000	$ 1,500,000	$ 1,500,000	$ 1,500,000	$ 1,500,000	$ 1,500,000	$ 1,500,000	$ 1,500,000	**$ 30,000,000**
Manhole Rehabilitation	$ 300,000	$ 324,000	$ 324,000	$ 324,000	$ 324,000	$ 324,000	$ 324,000	$ 324,000	$ 324,000	$ 324,000	$ 324,000	$ 324,000	$ 54,000	$ 54,000	$ 54,000	$ 54,000	$ 54,000	$ 54,000	$ 54,000	$ 54,000	**$ 4,296,000**
Total Sewer Rehabilitation	$ 1,800,000	$ 1,824,000	$ 1,824,000	$ 1,824,000	$ 1,824,000	$ 1,824,000	$ 1,824,000	$ 1,824,000	$ 1,824,000	$ 1,824,000	$ 1,824,000	$ 1,824,000	$ 1,554,000	$ 1,554,000	$ 1,554,000	$ 1,554,000	$ 1,554,000	$ 1,554,000	$ 1,554,000	$ 1,554,000	**$ 34,296,000**
Southeast Interceptor																					
Design (5% of construction)	$ 519,582																				**$ 519,582**
Replace Fox Point Pump Station		$ 3,000,000																			**$ 3,000,000**
Replace Fox Point Force Main			$ 1,437,500																		**$ 1,437,500**
30" interceptor from Fox Point to Burr Oak			$ 588,800																		**$ 588,800**
30" interceptor from Burr Oak to West Ave				$ 2,137,850																	**$ 2,137,850**
21" sewer to Les Paul Pkwy & Legend Hill Ln					$ 1,624,388																**$ 1,624,388**
8" sewer from Interceptor to Milky Way PS					$ 65,550																**$ 65,550**
21" Interceptor to Heyer Dr. PS						$ 1,537,550															**$ 1,537,550**
Southeast Interceptor Total	$ 519,582	$ 3,000,000	$ 2,026,300	$ 2,137,850	$ 1,689,938	$ 1,537,550															**$ 10,911,220**
West-Side Interceptor (Route 2)																					
Design (5% of construction)	$ 1,421,133																				**$ 1,421,133**
Build 42" from intersection of Les Paul Pkwy. & MacArthur Rd. to Intersection of MacArthur Rd. & Commanche Ln.							$ 7,309,136														**$ 7,309,136**
Build 42" interceptor from MacArthur Rd. PS to Coneview PS along CR 71								$10,882,501													**$ 10,882,501**
Build 30" Interceptor to to Madison St. PS from Coneview PS								$ 169,650													**$ 169,650**
Build 30" interceptor to Fiddler's Creek PS								$ 658,365													**$ 658,365**
Build 30" sewer to Summit Ave PS									$ 1,384,387												**$ 1,384,387**
Build 30" sewer toTallgrass PS Tie In									$ 22,548												**$ 22,548**
Build 8" sewer to Tall Grass PS									$ 300,690												**$ 300,690**
Upsize Sewer from Interceptor Tie in to WWTP										$ 962,910											**$ 962,910**
Build 24" sewer to Pebble Valley											$ 2,074,886										**$ 2,074,886**
Build 24" sewer from interceptor through Coneview PS to Greenmeadow PS												$ 4,657,594									**$ 4,657,594**
West-Side Interceptor Total	$ 1,421,133						$ 7,309,136	$11,710,516	$ 1,707,626	$ 962,910	$ 2,074,886	$ 4,657,594									**$ 29,843,801**
West Avenue Interceptor																					
Design (5% of construction for pipes, 15% for PS)				$ 140,520	$ 228,420			$ 121,380	$ 170,940												**$ 661,260**
24-inch Sewer					$ 2,810,400																**$ 2,810,400**
Lift Station #2						$ 1,522,800															**$ 1,522,800**
18-inch Mill Creek Sewer									$ 2,427,600												**$ 2,427,600**
18-inch Pebble Brook Sewer										$ 927,600											**$ 927,600**
Lift Station #1										$ 830,400											**$ 830,400**
West Avenue Interceptor Total				$ 140,520	$ 3,038,820	$ 1,522,800	$ -	$ 121,380	$ 2,598,540	$ 1,758,000											**$ 9,180,060**
Pump Stations & Force Mains																					
Wesley Drive PS & FM Elimination	$ 130,000																				**$ 130,000**
Pump Station Flood Protection (Sunset Dr, Coneview, Summit Ave, and West Ave)	$ 200,000																				**$ 200,000**
Bluemound (Control panel, valve vault)	$ 200,000																				**$ 200,000**
Badger Drive plumbing upgrade, pumps, piping, and valves moved out of wetwell)			$ 100,000																		**$ 100,000**
General Electric (Under way)	$ 330,000																				**$ 330,000**
Hollidale (Valve vault & control panel)		$ 200,000																			**$ 200,000**
Greenmeadow (Replace 2 pumps, re-wind motors, internal plumbing, force main piping, asbestos removal)			$ 50,000			$ 150,000															**$ 200,000**
Pearl (Electrical upgrade)					$ 50,000																**$ 50,000**
MacArthur (Control panel, valve vault)		$ 300,000																			**$ 300,000**
Fox Point (Control panel)		$ 50,000																			**$ 50,000**
Golf Road (Increase capacity / replace)				$ 350,000																	**$ 350,000**
Northview (Hatch, repair drive, force main)	$ 75,000																				**$ 75,000**
Woodfield (New wet well, valve vault)				$ 250,000																	**$ 250,000**
Walmart (Upgrade control panel)			$ 50,000																		**$ 50,000**
Patricia (Generator)					$ 50,000																**$ 50,000**
Pebble Valley (Generator)				$ 200,000																	**$ 200,000**
Other Pump Station Upgrades / Rehab ($40/yr/gpm)							$ 250,000	$ 250,000	$ 250,000	$ 250,000	$ 250,000	$ 250,000	$ 250,000	$ 250,000	$ 250,000	$ 250,000	$ 250,000	$ 250,000	$ 250,000	$ 250,000	**$ 3,500,000**
Replace Ferrous Force Mains																					
Ruben Drive												$ 1,135,000									**$ 1,135,000**
Northview													$ 130,000								**$ 130,000**
Wal-Mart													$ 220,000								**$ 220,000**
Sunset Drive														$ 700,000							**$ 700,000**
Badger Drive															$ 240,000						**$ 240,000**
MacArthur Road															$ 415,000						**$ 415,000**
Springbrook																$ 740,000					**$ 740,000**
Hollidale																		$ 13,000			**$ 13,000**
Woodfield																		$ 130,000			**$ 130,000**
Bluemound																		$ 94,000			**$ 94,000**
Total Pump Stations & Force Mains	$ 935,000	$ 550,000	$ 200,000	$ 800,000	$ 100,000	$ 150,000	$ 250,000	$ 250,000	$ 250,000	$ 250,000	$ 250,000	$ 1,385,000	$ 600,000	$ 950,000	$ 905,000	$ 990,000	$ 250,000	$ 487,000	$ 250,000	$ 250,000	**$ 10,052,000**
CMOM																					
Manhole Inspections	$ 56,000	$ 56,000	$ 56,000	$ 56,000	$ 24,000	$ 24,000	$ 24,000	$ 24,000													**$ 320,000**
CCTV	$ 68,640	$ 76,427	$ 52,765	$ 51,237	$ 51,143	$ 51,110	$ 53,491	$ 61,612													**$ 466,425**
Flusher Truck with Camera		$ 200,000																			**$ 200,000**
Camera Truck				$ 200,000																	**$ 200,000**
CMOM Total	$ 124,640	$ 332,427	$ 108,765	$ 307,237	$ 75,143	$ 75,110	$ 77,491	$ 85,612	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	**$ 1,186,425**
Collection System Total	**$ 4,800,355**	**$ 5,706,427**	**$ 4,159,065**	**$ 5,209,607**	**$ 6,727,901**	**$ 5,109,460**	**$ 9,460,627**	**$13,991,508**	**$ 6,380,166**	**$ 4,794,910**	**$ 4,148,886**	**$ 7,866,594**	**$ 2,154,000**	**$ 2,504,000**	**$ 2,459,000**	**$ 2,544,000**	**$ 1,804,000**	**$ 2,041,000**	**$ 1,804,000**	**$ 1,804,000**	**$ 95,469,506**

Table 18 – 20-Yr Capital Improvement Plan (No PS Elimination)

	2011	2012	2013	2014	2015	2016	2017	2018	2019	2020	2021	2022	2023	2024	2025	2026	2027	2028	2029	2030	Total
Collection System																					
Sewer Rehabilitation																					
Sanitary Sewer Rehabilitation - Minor Sewers	$ 1,500,000	$ 1,500,000	$ 1,500,000	$ 1,500,000	$ 1,500,000	$ 1,500,000	$ 1,500,000	$ 1,500,000	$ 1,500,000	$ 1,500,000	$ 1,500,000	$ 1,500,000	$ 1,500,000	$ 1,500,000	$ 1,500,000	$ 1,500,000	$ 1,500,000	$ 1,500,000	$ 1,500,000	$ 1,500,000	**$ 30,000,000**
Manhole Rehabilitation	$ 300,000	$ 324,000	$ 324,000	$ 324,000	$ 324,000	$ 324,000	$ 324,000	$ 324,000	$ 324,000	$ 324,000	$ 324,000	$ 324,000	$ 54,000	$ 54,000	$ 54,000	$ 54,000	$ 54,000	$ 54,000	$ 54,000	$ 54,000	**$ 4,296,000**
Total Sewer Rehabilitation	$ 1,800,000	$ 1,824,000	$ 1,824,000	$ 1,824,000	$ 1,824,000	$ 1,824,000	$ 1,824,000	$ 1,824,000	$ 1,824,000	$ 1,824,000	$ 1,824,000	$ 1,824,000	$ 1,554,000	$ 1,554,000	$ 1,554,000	$ 1,554,000	$ 1,554,000	$ 1,554,000	$ 1,554,000	$ 1,554,000	**$ 34,296,000**
West Avenue Interceptor																					
Design (5% of construction for pipes, 15% for PS)				$ 140,520	$ 228,420			$ 121,380	$ 170,940												**$ 661,260**
24-inch Sewer					$ 2,810,400																**$ 2,810,400**
Lift Station #2						$ 1,522,800															**$ 1,522,800**
18-inch Mill Creek Sewer									$ 2,427,600												**$ 2,427,600**
18-inch Pebble Brook Sewer										$ 927,600											**$ 927,600**
Lift Station #1										$ 830,400											**$ 830,400**
West Avenue Interceptor Total				$ 140,520	$ 3,038,820	$ 1,522,800	$ -	$ 121,380	$ 2,598,540	$ 1,758,000											**$ 9,180,060**
Pump Stations & Force Mains																					
Wesley Drive PS & FM Elimination	$ 130,000																				**$ 130,000**
Pump Station Flood Protection (Sunset Dr, Coneview, Summit Ave, and West Ave)	$ 200,000																				**$ 200,000**
Bluemound (Control panel, valve vault)	$ 200,000																				**$ 200,000**
Badger Drive plumbing upgrade, pumps, piping, and valves moved out of wetwell)			$ 100,000																		**$ 100,000**
General Electric (Under way)	$ 330,000																				**$ 330,000**
Hollidale (Valve vault & control panel)		$ 200,000																			**$ 200,000**
Greenmeadow (Replace 2 pumps, re-wind motors, internal plumbing, force main piping, asbestos removal)			$ 50,000			$ 150,000															**$ 200,000**
Pearl (Electrical upgrade)					$ 50,000																**$ 50,000**
MacArthur (Control panel, valve vault)		$ 300,000																			**$ 300,000**
Fox Point (Control panel)		$ 50,000																			**$ 50,000**
Golf Road (Increase capacity / replace)				$ 350,000																	**$ 350,000**
Northview (Hatch, repair drive, force main)	$ 75,000																				**$ 75,000**
Woodfield (New wet well, valve vault)				$ 250,000																	**$ 250,000**
Walmart (Upgrade control panel)			$ 50,000																		**$ 50,000**
Patricia (Generator)					$ 50,000																**$ 50,000**
Heyer Drive (Generator, roof, windows)					$ 300,000																**$ 300,000**
Pebble Valley (Generator)				$ 200,000																	**$ 200,000**
Replace West Ave PS (8 MGD)						$ 270,000	$ 1,800,000														**$ 2,070,000**
Other Pump Station Upgrades / Rehab ($40/yr/gpm)							$ 500,000	$ 500,000	$ 500,000	$ 500,000	$ 500,000	$ 500,000	$ 500,000	$ 500,000	$ 500,000	$ 500,000	$ 500,000	$ 500,000	$ 500,000	$ 500,000	**$ 7,000,000**
Replace Ferrous Force Mains																					
West Avenue (Upsize to 18 inches)							$ 550,000														**$ 550,000**
Greenmeadow								$ 1,060,000													**$ 1,060,000**
Pebble Valley									$ 760,000												**$ 760,000**
Heyer Drive										$ 250,000											**$ 250,000**
Burr Oak											$ 1,010,000										**$ 1,010,000**
Coneview										$ 470,000											**$ 470,000**
Ruben Drive												$ 1,135,000									**$ 1,135,000**
Northview													$ 130,000								**$ 130,000**
Milky Way													$ 235,000								**$ 235,000**
Wal-Mart													$ 220,000								**$ 220,000**
Sunset Drive														$ 700,000							**$ 700,000**
Badger Drive															$ 240,000						**$ 240,000**
MacArthur Road															$ 415,000						**$ 415,000**
Springbrook																$ 740,000					**$ 740,000**
Summit																	$ 425,000				**$ 425,000**
Hollidale																		$ 13,000			**$ 13,000**
Woodfield																		$ 130,000			**$ 130,000**
Bluemound																		$ 94,000			**$ 94,000**
Total Pump Stations & Force Mains	$ 935,000	$ 550,000	$ 200,000	$ 800,000	$ 400,000	$ 420,000	$ 2,850,000	$ 1,560,000	$ 1,260,000	$ 1,220,000	$ 1,510,000	$ 1,635,000	$ 1,085,000	$ 1,200,000	$ 1,155,000	$ 1,240,000	$ 925,000	$ 737,000	$ 500,000	$ 500,000	**$ 20,682,000**
CMOM																					
Manhole Inspections	$ 56,000	$ 56,000	$ 56,000	$ 56,000	$ 24,000	$ 24,000	$ 24,000	$ 24,000													**$ 320,000**
CCTV	$ 68,640	$ 76,427	$ 52,765	$ 51,237	$ 51,143	$ 51,110	$ 53,491	$ 61,612													**$ 466,425**
Flusher Truck with Camera		$ 200,000																			**$ 200,000**
Camera Truck				$ 200,000																	**$ 200,000**
CMOM Total	$ 124,640	$ 332,427	$ 108,765	$ 307,237	$ 75,143	$ 75,110	$ 77,491	$ 85,612	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	**$ 1,186,425**
Collection System Total	**$ 2,859,640**	**$ 2,706,427**	**$ 2,132,765**	**$ 3,071,757**	**$ 5,337,963**	**$ 3,841,910**	**$ 4,751,491**	**$ 3,590,992**	**$ 5,682,540**	**$ 4,802,000**	**$ 3,334,000**	**$ 3,459,000**	**$ 2,639,000**	**$ 2,754,000**	**$ 2,709,000**	**$ 2,794,000**	**$ 2,479,000**	**$ 2,291,000**	**$ 2,054,000**	**$ 2,054,000**	**$ 65,344,485**

CHAPTER VIII – FINAL RECOMMENDATIONS/CONCLUSIONS

In conjunction with this Master Planning Report, Donohue has prepared a Capacity, Maintenance, Operations, and Management (CMOM) Report. This report lays out managerial and operational modifications that will enable Waukesha to improve system reliability as cost-effectively as possible.

This study has determined that the Waukesha collection system generally has adequate capacity to convey peak wet and dry weather flows. However the Pebble Valley and Greenmeadow pump stations are likely to become overloaded during severe events. While the proposed West-Side Interceptor would eliminate these bottlenecks, we recommend Waukesha consider the cost of upgrading these stations and their force mains as part of a cost-benefit analysis of the proposed interceptor.

Force mains have historically been the most failure-prone element of the collection system. Waukesha has replaced failed force mains, and the desktop study performed under Phase I of this study and the subsequent testing performed under Phase II have determined that the risk of failure has been significantly reduced. However, since visually inspecting these facilities is not practical, uncertainty remains as to their reliability. Donohue recommends eliminating the remaining ferrous force mains as conditions permit.

Overall, I/I entering Waukesha's sanitary sewer system on an annual average basis is not excessive. However flow monitoring has identified pockets where it might be cost-effective to reduce I/I; these include the areas upstream of the Heyer Drive and Pebble Valley pump stations.

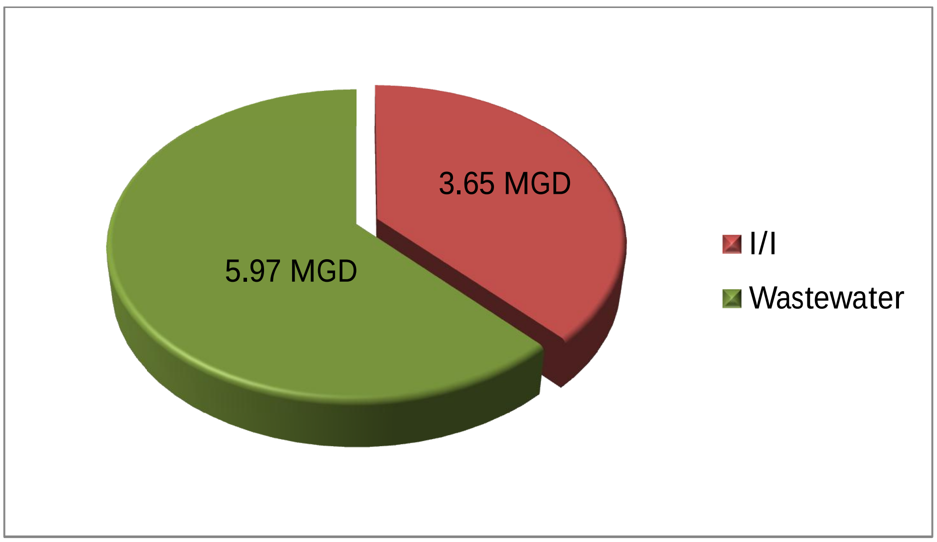


Figure 33 – WWTP Flow Components

While Pebble Valley experiences a wet weather response indicative of direct inflow, smoke testing detected relatively few defects. The next most likely source of the inflow is sump pumps. We recommend Waukesha inspect a representative sample of homes to ascertain the extent to which sump pumps are contributing I/I. If sump pumps are found to be a significant contributor of I/I, Waukesha may want to enforce its ordinance prohibiting these connections. If wet weather flows in Pebble Valley are not reduced, Waukesha will have to convey these flows to the plant; if the West-Side interceptor is not constructed, the Pebble Valley and possibly Greenmeadow pump stations will have to have their capacities increased to provide the 25-year level of protection.

There are no obvious potential sources of I/I in the Heyer Drive area. Waukesha's ongoing sewer televising will likely be sufficient to locate the source of the clear water flow.

While smoke testing revealed several significant defects in the 2,000-acre downtown area, repairing these is unlikely to significantly reduce the quantity of I/I flow monitoring indicates this area generates. The average age of these sewers is 70 years. I/I is likely the natural consequence of pipeline deterioration. Waukesha's ongoing sewer televising and CMOM programs should enable Waukesha to reduce I/I to what can be cost-effectively achieved.